AN EXPERIMENTAL INVESTIGATION OF THE LOAD
CARRYING CAPACITY OF DRILLED CAST-IN-PLACE
CONCRETE PILES BY MEANS OF STATIC LOAD TESTS

P. Kozicki.

April, 1959.

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AN EXPERIMENTAL INVESTIGATION OF THE LOAD

CARRYING CAPACITY OF DRILLED CAST-IN-PLACE

CONCRETE PILES BY MEANS OF STATIC LOAD TESTS.

### A DISSERTATION

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR

THE DEGREE OF MASTER OF SCIENCE.

FACULTY OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING

BY

PETER KOZICKI, B.Sc.

EDMONTON, ALBERTA
April, 1959

# UNIVERSITY OF ALBERTA FACULTY OF GRADUATE STUDIES

The undersigned hereby certify that they have read and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled

AN EXPERIMENTAL INVESTIGATION OF THE LOAD CARRYING CAPACITY OF

DRILLED CAST-IN-PLACE CONCRETE PILES

BY MEANS OF STATIC LOAD TESTS

Submitted by PETER KOZICKI, B. Sc. in partial fulfilment of the requirements for the degree of

MASTER OF SCIENCE



#### ABSTRACT

The purpose of this investigation was to determine the ultimate carrying capacity of drilled cast-in-place concrete piles in typical Edmonton soils. The investigation was divided into two parts, the first part was primarily to check the design capacity of one of the drilled cast-in-place concrete piles used in the foundation for the Proposed Chemistry and Physics Building. The second part consisted of attempting to check the theoretical values of skin friction and end-bearing based on laboratory test results against the values as determined by means of actual pile load tests. Additional tests were also made to check the theoretical capacity based on laboratory test results of a combined skin friction and end-bearing pile.

Based on the field test results, it was found that:

- (a) the pile that is part of the foundation for the Chemistry and Physics Building appears to have a factor of safety somewhat greater than 3.
- (b) the test results on the friction pile were considerably higher than expected, possibly due to the tangential adfreezing strength of the soil.
- (c) test results of the end-bearing pile check with the theoretical analysis.
- (d) in the case of the combined skin friction and end-bearing pile, it appears that the tangential adfreezing strength of the soil affected the test results to a certain extent.

#### **ACKNOWLEDGMENTS**

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# CHAPTER I

## INTRODUCTION

### HISTORY

Since prehistoric times piles and pile foundations have been in common use as a means of support wherever the ground appeared incapable of sustaining the pressure exerted by the footings. Previous to the 19th century there was little or no basis for design of a pile foundation. As timber was abundant and labor cheap, as many piles were driven as the ground would permit. Settlement caused no concern as the prevalent type of structure could withstand a considerable amount of unequal settlement without injury.

In the 19th century, when industrial development created a demand for heavy but inexpensive structures in locations underlain by soft ground, the cost of pile foundations became an item of consequence, and engineers were expected to specify no more piles than were necessary to provide adequate support for the buildings. Consideration was also given to the use of steel, concrete, and composite piles of various types, which in comparison with timber piles, would provide a more permanent type of foundation as well as being able to support larger loads per pile. In order to be able to specify the most economical type of pile foundation, an engineer had to have some knowledge of the ultimate load that an individual pile could carry. Efforts to obtain the necessary information at a minimum expenditure of money and labor led to theoretical speculations that resulted in an impressive assortment of dynamic pile formulas. However, the realization slowly grew that the dynamic pile formulas had inherent shortcomings, and it became more

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and more customary to determine the allowable load per pile on all but the smallest jobs by making load tests on test piles.

For piles such as the 'Drilled Cast-in-Place Concrete Piles,"
where a dynamic method is not used in constructing the pile, the
method of determining the allowable load per pile is a subject of
considerable controversy among engineers. This type of pile was
first used in the 1920's and the method originally employed in
constructing the pile was to drill holes with a hand auger and fill
them with concrete. The hand augers were later modified to use
mechanical methods to turn the augers. These methods have today been
replaced by power machinery capable of drilling holes from 16 inches
to 96 inches in diameter, and to depths of 60 feet or more. In highly
plastic clay soils the holes will normally stay open without casing
until filled with concrete.

# THEORY OF PILE ACTION

A pile transfers load into the surrounding soil by either (a) friction along the embedded length of the pile, (b) point-bearing, or, (c) combination of point-bearing and friction. Piles may be classified roughly as "friction" or "end-bearing", according to the manner in which they develop support. In any case, the load must be carried ultimately by the soil layers around and below the points of the piles, and an accurate knowledge of the compressibility of these soil layers is of utmost importance in predicting the load the pile will support.

### FRICTION PILES

Friction piles in cohesive soils develop their carrying capacity from tangential forces along the sides of the pile and it can be assumed

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that very little of the load is carried by the point. These tangential forces produce shearing stresses in the soil mass. The limiting value of the tangential resistance is therefore determined by the shearing strength of the soil in the vicinity of the pile, and by the frictional resistance between the soil and the surface of the pile. The points of such piles may carry some load because the soil at the elevation of the pile point is under restraint due to the weight of the soil above.

A full discussion on remolding and driving is given in (1) and (2).

Friction piles driven in loose granular material serve to densify
the material and as a result are some times referred to as 'Compaction
Piles". These piles are some times used to increase the relative density
of the sand thus increasing the bearing capacity of the sand.

# POINT-BEARING PILES

If a pile penetrates a stratum of soft soil onto a dense incompressible stratum, the load is carried entirely by end-bearing. Possibly
some of the load may initially be carried by skin friction, however, it
will eventually all be carried by end-bearing, (3). In some instances
if a pile passes through a very compressible soil, the pressure

<sup>\*</sup> Numbers in parenthesis refer to references contained in the bibliography.

transferred to this soil by skin friction will gradually consolidate it.

Consolidation of this soil will continue until all of the applied load is carried by the pile point and if the load assigned to the pile exceeds the point resistance, then considerable settling of the pile may occur.

COMBINED POINT-BEARING AND FRICTION

Between the two extreme cases of full point-bearing and full friction with no point-bearing, there is a variety of possible combinations of point-bearing and side resistance. It is difficult to determine how much load is carried by the point and how much by the sides. In addition, it is not known how the tangential forces are distributed along the length of the pile. The distribution of stresses surrounding the pile is also unknown. Various methods of mathematical analysis have been

applied to these problems but without much success to the present time.

If the pile is tapered, then in the case of skin friction piles in sandy soils, the friction on the sides is increased as displacement of the soil along the length of the pile must take place before any pile movement occurs. This has the effect of increasing the carrying capacity of the tapered pile in comparison with that of a parallel-sided pile having the same superficial area in contact with the surrounding soil. With end-bearing piles the effect is negligible, and the increased upper diameter fulfils no useful purpose from this standpoint.

# IMPORTANCE OF THE SOIL ACTION IN A PILE FOUNDATION

In evaluating any pile foundation it is necessary to evaluate the action of the particular soil type involved as well as the behavior of the pile itself. The properties of the common building materials are relatively well known, and the designer can rely upon their strength and performance. On the other hand the properties and behavior of the

when continuous samples are taken in a drill hole, only representative tests can be run to determine the soil properties and furthermore they may vary over the site investigated. Thus the designer and builder of a pile foundation must guard against possible variations of the soil properties on any particular site.

The total resistance of the soil to the penetration of a pile consists of:

- 1. The resistance that the soil has to displacement.
- 2. The resistance the soil has to a reduction or increase in the volume of voids in the soil adjacent to the pile.
- 3. The resistance of the soil to movement at the pile soil contact.

This resistance depends on the character and density of the soil and on the pressure exerted against the sides of the pile. In loose granular soil vibration of the pile driving operation causes a densification of the sand accompanied by a large increase in friction. Also in a loose granular soil the displacement at the point is practically all by reduction in the volume of voids. In saturated cohesive soils practically no volume changes occur, thus as the pile is driven bodily displacement of the soil takes place.

# SETTLEMENT

Failure load (4) of an individual pile is considered as that beyond which an increase in load produces a disproportionate increase in settlement. Since a disproportionate increase in settlement is caused by shear failure of the soil, this criteria must apply only to cohesive soils. In granular soils there may not be a load where a

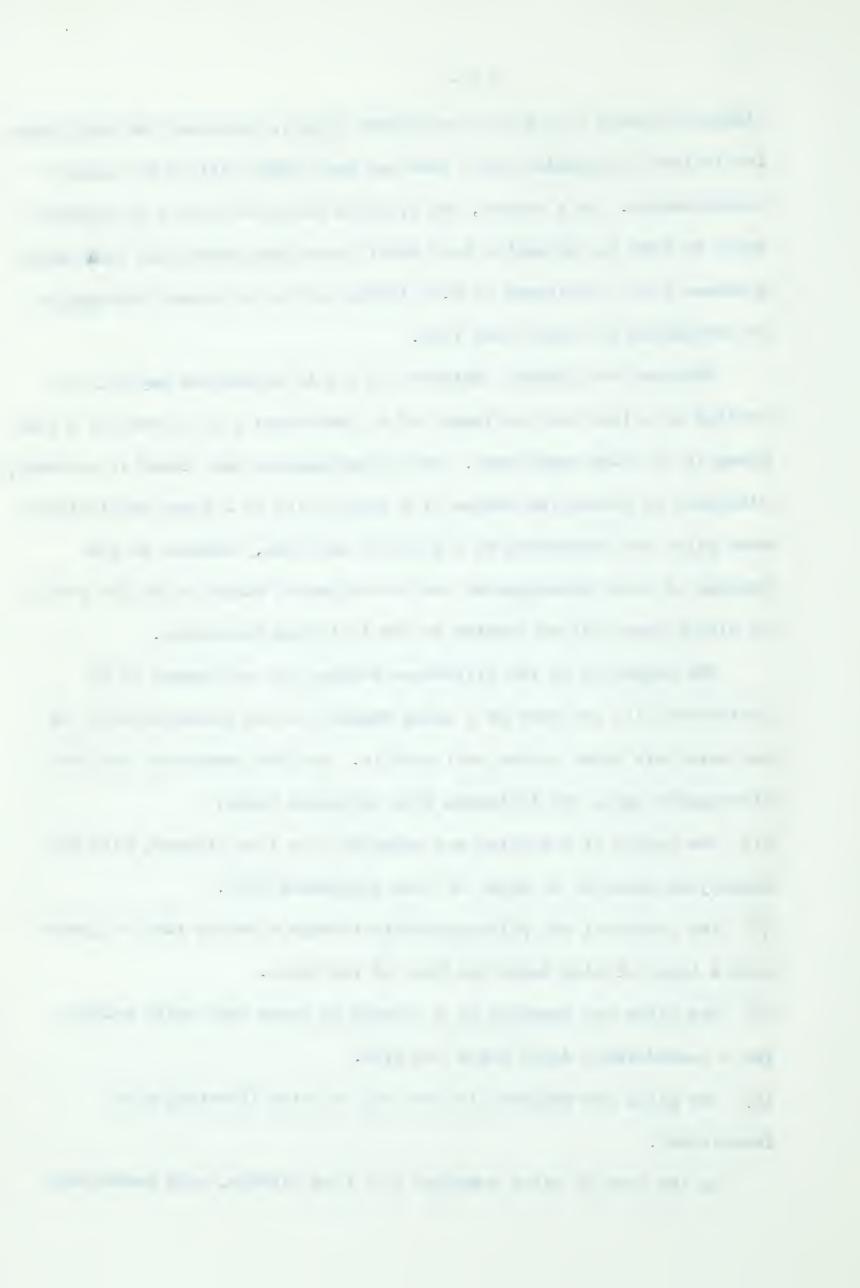
disproportionate increase in settlement occurs, therefore the definition for failure in granular soils does not mean shear failure but rather consolidation. As a result, the criteria for pile failure in granular soils is that the allowable load shall in no case exceed the load which produces a net settlement of 0.01 inches per ton of gross load applied as determined by a pile load test.

Whatever the general character of a pile foundation may be, the ability to relate the settlement of an individual pile to that of a pile group is of prime importance. Many investigators have found it extremely difficult to relate the action of a single pile to a group particularly when piles are surrounded by a granular material. However as the purpose of this investigation was the action of single piles the action of pile groups will be limited to the following discussion.

The magnitude of the difference between the settlement of an individual pile and that of a group depends on the characteristics of the materials shown in the soil profile. In this connection one must distinguish among the following four principal cases:

- (1) The points of the piles are embedded in a firm stratum, with the underlying material of equal or less compressibility.
- (2) The points of the piles penetrate through a bed of sand or gravel with a layer of clay below the base of the piles.
- (3) The piles are embedded in a stratum of loose sand which extends for a considerable depth below the pile.
- (4) The piles are embedded in soft silt or clay (floating pile foundation).

In the case of piles embedded in a firm stratum, with underlying



material of equal or less compressibility, very little settlement will occur. As a rule settlement ceases within a few months after all the loads are applied, provided the piles do not carry loads considerably in excess of their allowable loads.

For piles penetrating through a bed of cohesionless material with a layer of clay below the base of the piles, two different possibilities need be considered. If the pressure exerted on the clay by the foundation does not exceed the preconsolidation pressure, the influence of the beds of clay on the settlement of the foundation is unimportant. On the other hand if the pressure exerted by the weight of the building is great enough, the gradual consolidation of the layers of clay located beneath the points of the piles will cause settlement. The settlement is likely to progress at a decreasing rate during a period of many years. If the clay is soft and the layers thick, the ultimate settlement can be very large, although the factor of safety of the soil against failure may be fully adequate. For a given spacing of the piles and a given load per pile, the settlement increases with increasing size of loaded area.

In the case of piles embedded in a stratum of loose sand which extends for a considerable depth below the piles, a distinction has to be made between driving piles and drilled cast-in-place piles. As mentioned previously, driving piles densifies the surrounding sand which results in an increased skin friction. The ultimate bearing capacity of driven piles in sand increases roughly with the square of the depth of penetration (1). Large-scale experiments (5) have shown that compaction caused by driving one pile influences the bearing capacity of any other

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pile located within a distance equal at least to five times the diameter of the pile. As a consequence if only one pile in a group is loaded, its settlement under a given load will decrease as the number of piles increases. Nevertheless, if all the piles are loaded, the settlement of the group under a given load per pile increases with the number of piles. Cast-in-place piles formed by drilling in loose sand do not increase the relative density of the sand as a result there is no large increase in the ultimate capacity as compared to driven piles. The ultimate capacity is dependent on the actual condition of the sand and would not be increased by the installation of additional piles. The difference in settlement between a single pile and a group of piles, each under the same magnitude of load as the single pile, would roughly be the same as for the case of the driven piles. The difference would be that, under the same magnitude of loading, the driven piles would settle considerably less than the drilled cast-inplace piles.

In the case of piles being embedded in a soft stratum the pile foundation would serve the purpose of transferring load to a lower level. If the soil is fairly homogenous to great depth, the settlement of a pile foundation carrying a given load distributed over a given area decreases appreciably with increasing length of piles. However, should the construction of a pile foundation be in an area of newly placed fill, problems may arise due to negative skin friction acting on the piles. If the subsoil consists of loose sand or highly permeable and relatively incompressible soils, the effect of the fill on the piles can be disregarded. On the other hand, if the subsoil contains layers of

soft silt or clay, the presence of the fill considerably increases the load on the piles, and, as a consequence, also causes an increase in settlement. Before the piles are installed, the compressible strata gradually consolidate under the weight of the fill material. After the piles are installed it can no longer settle freely because its downward movement is resisted by skin friction between the fill and the piles. An imperceptible downward movement of the fill with respect to the piles is sufficient to transfer onto the piles the weight of the fill located within the cluster. If this load is greater than the point resistance of the pile, the settlement of the foundation will be excessive, regardless of what the ultimate bearing capacity a load test may indicate.

The problem of arriving at the bearing capacity of piles by means of laboratory test results is difficult and is practically impossible for the case of piles driven in sand. The most reliable procedure for determining the bearing capacity of an individual pile in sand is a static load test. This is not always practicable and other tests such as the standard tests or the cone penetration tests are used. With cohesive soils the static load test is the best test for determining the bearing capacity of an individual pile provided the long term settlement effect is taken into account. The standard penetration test or the cone penetration test is not reliable in cohesive soils and the reliability of laboratory strength test results depends on the uniformity of the soil conditions. This investigation was to serve the purpose of checking the design method using laboratory strength test results against the results of static load tests.

#### PILE LOAD TESTS

There have been innumerable arrangements (7) of apparatuses developed for making loading tests on piles. A great deal of flexibility in design and ingenuity has been applied to make tests with the greatest economy of time and the use of available equipment.

Test loads may be applied by (a) a direct load such as heavy weights or water tanks placed on a platform; (b) jacking against a loaded platform or against an existing structure; (c) the use of anchor piles.

Direct load can be applied by any convenient means such as pigiron, earth, water tanks, or precast concrete blocks. The procurement of sufficient fixed load is sometimes difficult, and the removal of such full load for repeated loadings and releases on the same pile, which is usually desirable, is nearly impracticable. Water tanks may be arranged for draining and refilling fairly readily. There is danger from improperly fixed loads. Corner supports should be placed close under loaded platforms to catch the load should tilting occur because of shifting of the load or yielding of the soil. Jacking against fixed load on platforms is preferable to resting the load on the pile. The platforms always remain resting on cribbing, thus eliminating the danger of the platforms tilting.

Jacking is usually done with hydraulic jacks employing a gas or liquid under pressure. Another advantage of jacking is that the loads can be applied and released quickly and at will, permitting quick determination of the net settlement of the pile, or movement in the soil, after rebound has occurred.

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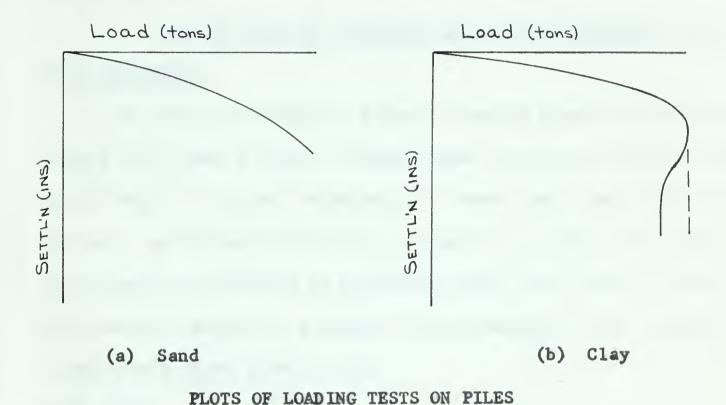
A compact set-up is obtained through the use of anchor piles as it does away with the cumbersome weights required for the fixed loads. Two anchor piles at least 5 feet on each side of the test pile may be used. Jacking is done against an I-beam which is fastened rigidly to the anchor piles. The anchor piles should be larger in diameter than the test pile and they should extend deeper into the ground in order to provide enough reaction to test the test pile to failure. Loads can be applied and removed simply by controlling the pressure on the jack.

Test loadings on piles are the final answer for determining the ultimate capacity of piles in sand. However for piles in clay, a short term test such as a pile load test is not entirely suitable as the cohesive strata cannot obtain its full settlement until after soil consolidation, which may take years. If the pile is in such a soil the ultimate compression cannot be determined, however the shearing value can be determined. Furthermore, the settlement of small loaded area of cohesive soil would have no relation to the much greater settlement which would occur under the same pile if it were one of many under the entire structure. It is impossible to evaluate tests unless adequate boring records present a complete picture of the soil conditions. Satisfactory pile load tests can be made in any soil through which water can seep freely in the voids.

Because of the difference in the permeability of clay and sandy soils, the Load vs Settlement Curves for the two types of soils have distinct characteristics. As shown below the curve for a pile loading

\* . . .  test in sand has a continuously increasing slope while in a test on clay the plot may be practically a straight line nearly to failure.

After reaching its peak value, it will then drop back.



Therefore in the interpretation of test pile results it is highly important to have a complete understanding of the actual soil conditions as well as an appreciation of the group action of piles.

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#### CHAPTER II

#### DESCRIPTION OF TEST PILES

Typical Sections of piles tested are shown on Plate 1 and are as follows:

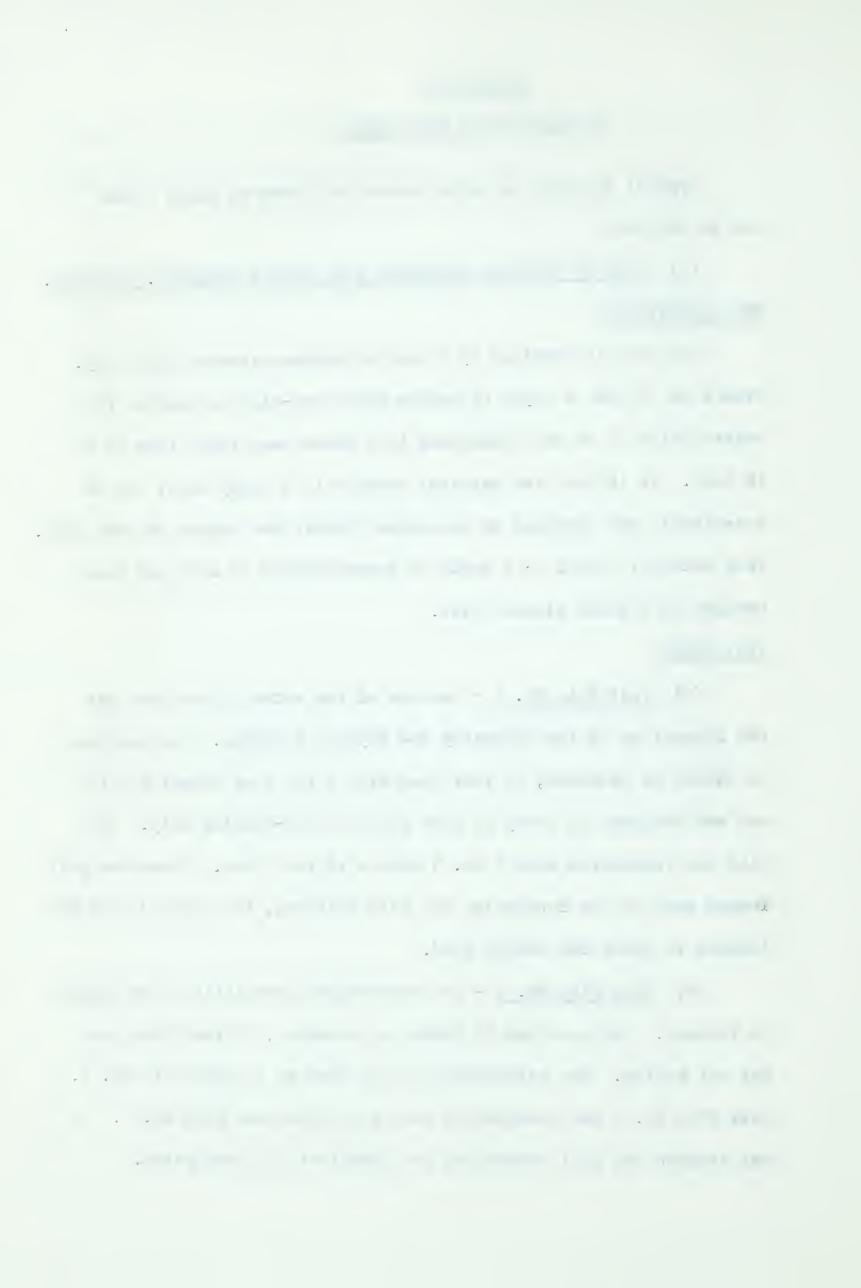
(1) SITE OF PROPOSED CHEMISTRY AND PHYSICS BUILDING. Plate 1A.
SOIL CONDITIONS

The site is overlain by 6 feet of medium plastic stiff clay.

From 6 to 15 feet a layer of medium dense non-plastic glacial till exists which is in turn underlain by a dense sand layer from 15 to 18 feet. At 18 feet the material changes to a sandy silt, low to non-plastic and contains an occasional pebble and layers of clay till. This material exists to a depth of approximately 25 feet and then changes to a dense glacial till.

## TEST PILES

- (a) Test Pile No. 1 was one of the actual piles used for the foundation of the Chemistry and Physics Building. The pile was 12 inches in diameter, 23 feet long with a two foot diameter bell and was designed to carry 25 tons based on end-bearing only. The pile was reinforced with 4 No. 5 bars x 10 feet long. Since the pile formed part of the foundation for this building, the applied load was limited to twice the design load.
- (b) Test Pile No. 2 was constructed especially to be tested to failure. The pile was 12 inches in diameter, 23 feet long and was not belled. The reinforcing is the same as in Test Pile No. 1. Test Pile No. 2 was constructed four feet from Test Pile No. 1. It was assumed the soil conditions are identical for both piles.



# (II) SITE OF WEST-END CITY YARD. Plate 1B.

#### SOIL CONDITIONS

The site is overlain from zero to 1 foot by a layer of gravel. From 1 foot to 21 feet below ground surface a deposit of firm highly plastic clay exists which is underlain from 21 feet to 22 feet by a layer of soft clayey silt. A deposit of firm to dense glacial till exists between 23 feet and 30 feet.

### TEST PILES

- (a) <u>Test Pile No. 3</u> was a 12 inch diameter pile, 26 feet long, and was not belled. No vertical reinforcing was used in the pile and it was to be loaded to failure.
- (b) Test Pile No. 4 was a 12 inch diameter pile, 23 feet long, no bell, and was suspended three feet above the bottom of the hole by means of a one inch thick steel plate which was held in position by four No. 5 rods. The rods were welded to the plate, and in order to support the plate in place while the concrete set up, the rods were welded to a five inch deep channel which in turn was supported at either end on 12 inch x 12 inch timbers. To prevent any concrete from passing through between the outside of the plate and the hole a 1/4 inch thick membrane, 14 inches in diameter was fastened to top of the 12 inch diameter steel plate, before lowering the plate down the hole. Test Pile No. 4 was constructed especially to determine the ultimate value of skin friction in a typical highly plastic Edmonton clay soil.
- (c) <u>Test Pile No. 5</u> was a 12 inch diameter pile 26 feet deep, no bell. It was formed by using a 12 inch diameter steel casing

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Photo 1 - Test Pile No. 5 - 12 inch diameter steel casing.



were welded at 8 foot centres on the outside of the steel casing to prevent any buckling which might occur during the testing of the pile. The outside diameter of the collars was 16 inches, which was the diameter of the drilled hole. The function of the flare at the bottom end was to give a confining effect to the soil just below the bottom of the pile. This pile was constructed to determine the effect of endbearing, and was to be loaded to failure.

Since the time available to carry out this investigation was limited, 3,000 lb. high early strength concrete was used in all piles with the exception of Test Pile No. 1, where standard 3,000 lb. concrete was used.

All piles were tested in compression and no reinforcing was required in any of the piles with the exception of Test Pile No. 4, where the reinforcing was used to support the steel plate above the bottom of the hole.

The piles were drilled with a Hughes Williams Power Auger and the bell in Test Pile No. 1 was formed by means of a mechanical belling tool. Immediately after completion of drilling, concrete was poured into the hole, and the top of the pile was carefully formed in order to provide a level bearing surface.

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#### CHAPTER III

#### TEST SETUP

It was felt that a pile setup carrying 70 ton of pig-iron kentledge would be suitable to test any of the piles to failure, if so desired. Economics was also a governing factor in limiting the test pile setup to 70 tons.

The test setup is shown in Photo 2 and Plate 2. A 175 ton hydraulic jack was used to transfer the dead load onto the pile. The amount of load transferred was recorded by means of the hydraulic jack which had been calibrated to 100 tons. Calibration test results are shown in Table 1 and the calibration curve is shown on Plate 3.

The pig-iron was supported on a 14 foot square platform and a 16 WF 36 I-beam was used to take up the load. The bottom of the I-beam was supported on a frame work of 12 inch x 12 inch timbers, four feet above ground surface. This provided sufficient room for placing of the jack on the top of the pile. A bearing plate 2 inches thick was used between the top of the pile and the jack, as shown on Plate 2. This bearing plate was necessary in order to prevent crushing of the concrete at the top of the pile.

A 5 inch channel supported at either end, was used as a reference point for the extensometer readings. For Test Pile No. 1, the supports used were one inch diameter rods, four feet long, driven two feet into the ground at a distance of 10 feet from the centre of the pile. This did not prove satisfactory as it was found that when over 37 tons of load was transferred to the pile, the extensometer readings did not agree with the level readings. If the load on the pile was less than

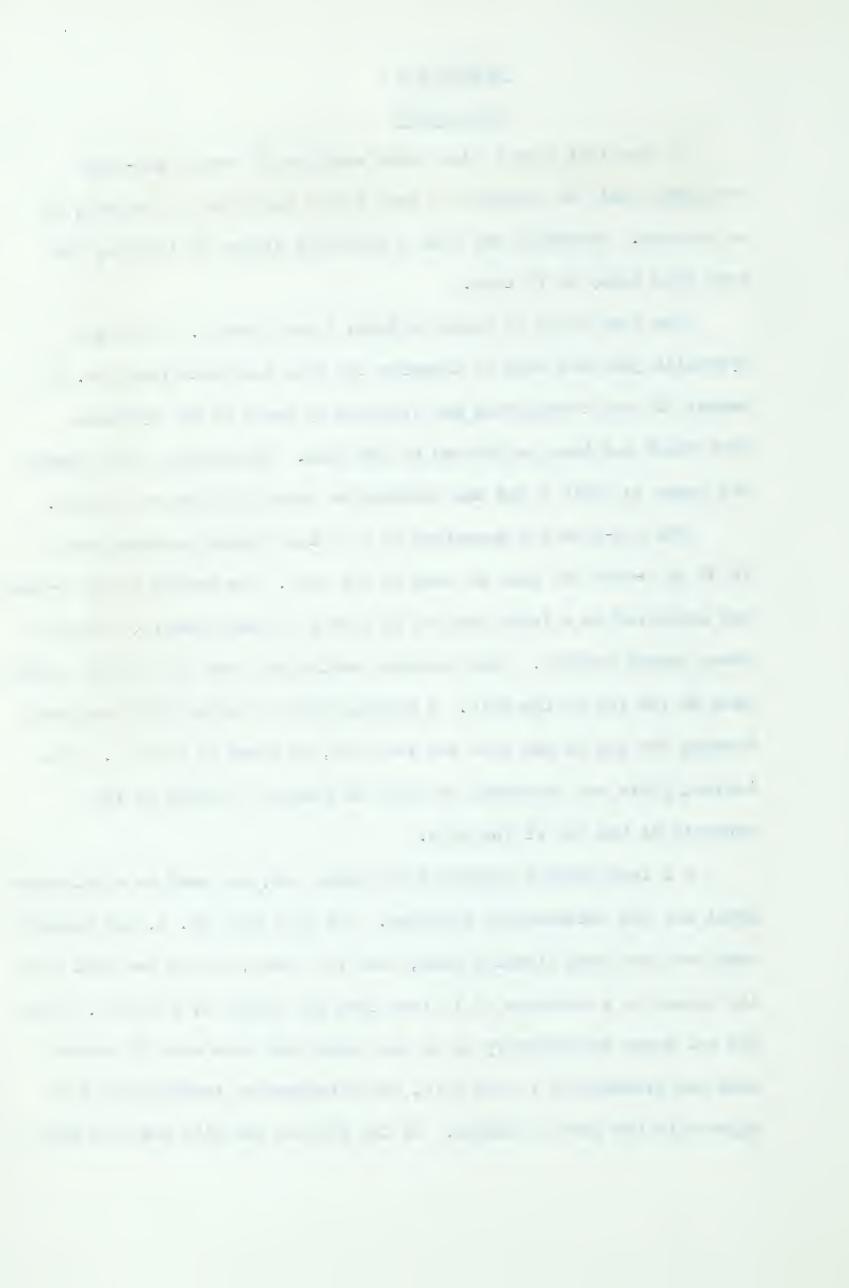




Photo 2 - Test Setup.



37 tons, then the two readings were in agreement. As a result it was felt that, as the load was being transferred from the timber crib to the pile, the ground tended to rebound. Since the ground was frozen at the time that the test was being performed, it is conceivable that this was occurring. In order to get around this problem it was decided to drive a two inch diameter pipe 4 feet into the ground. The rod used to support the channel was driven inside this pipe, Photo 3. This method proved satisfactory for fixing the channel for the balance of the testing program.

In order to check the extensometer readings a surveyor's level and target rod were used. The level was set up at approximately 50 feet from the test setup and a reference point at approximately the same distance away was used as a bench mark. In all cases with the exception of the test on Test Pile No. 1, the extensometer and level readings were in close agreement. The reason for the discrepancy in the readings on Test Pile No. 1 were discussed previously.

A magnetic holder as shown on Plate 2 and Photo 4, was fixed on the jack to hold the extensometer in place during the testing period.

Test Pile No's 1 and 2 were constructed at a distance of 4 feet apart as were Test Pile No's 4 and 5, in order that both piles could be tested with one test setup. In both cases shifting of a portion of the pig-iron kentledge was all that was required, to obtain the desired reaction load. This reduced the cost of the investigation as well as speeding up the testing.

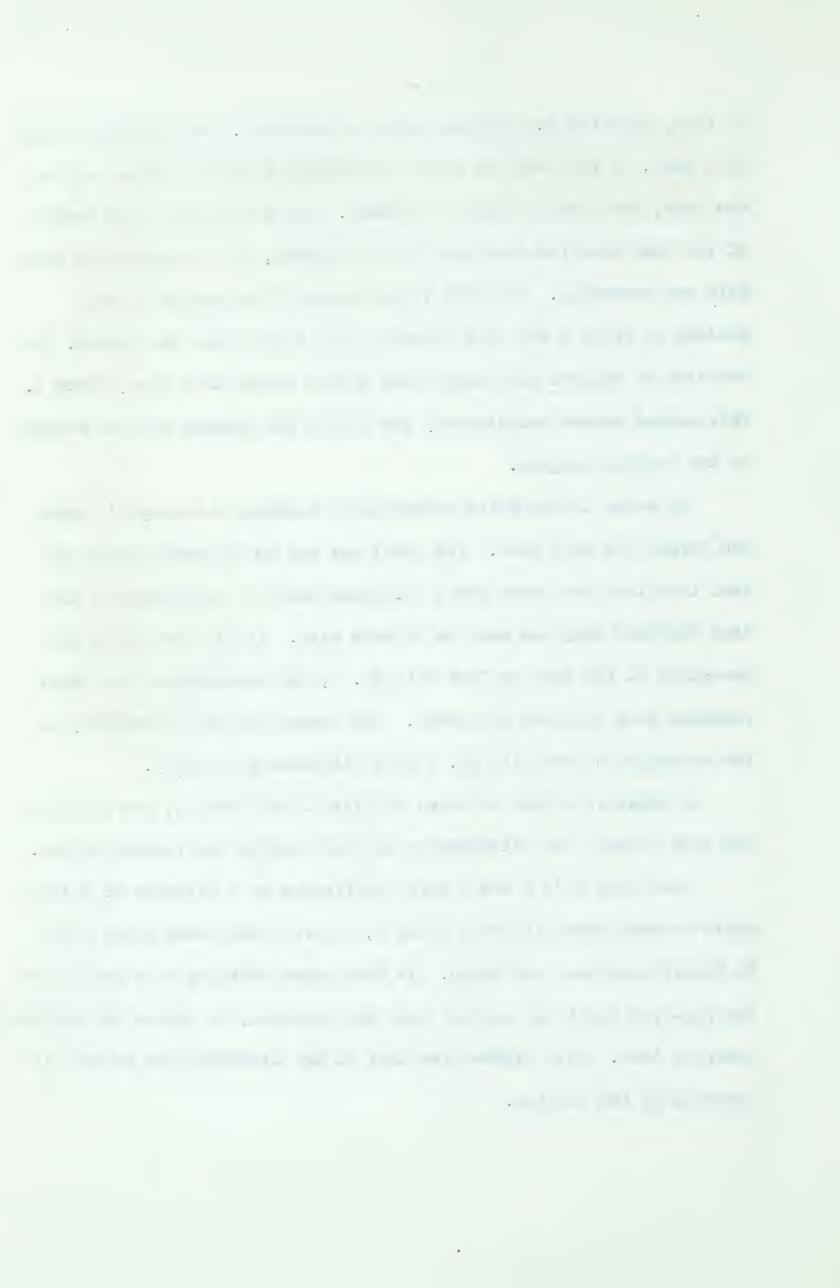




Photo 3 - Channel Support.





Photo 4 - Setup Used to Record Settlement.



#### CHAPTER IV

#### ANALYSIS OF SOIL CONDITIONS

The holes for the Test Piles were all drilled by means of a power auger. Undisturbed soil cores were recovered during drilling by pressing 2-inch and 3-inch diameter thin walled shelby spoons into the bottom of each hole at regular intervals as drilling progressed. Soil cores recovered in this manner were carefully sealed with Paraffin and taken to the Civil Engineering Building at the University where they were subsequently analysed. In addition disturbed samples were taken at regular intervals for natural moisture content determination.

## (1) SITE OF CHEMISTRY AND PHYSICS BUILDING

Detailed soil conditions together with field and laboratory test results for this site are given in Appendix A, and are summarized on Plate 2A.

No samples were taken in the drilling of Test Pile No. 1 and therefore since Test Pile No. 2 was only 4 feet away it was assumed that the test results of samples taken in the drilling of Test Pile No. 2 apply to both piles.

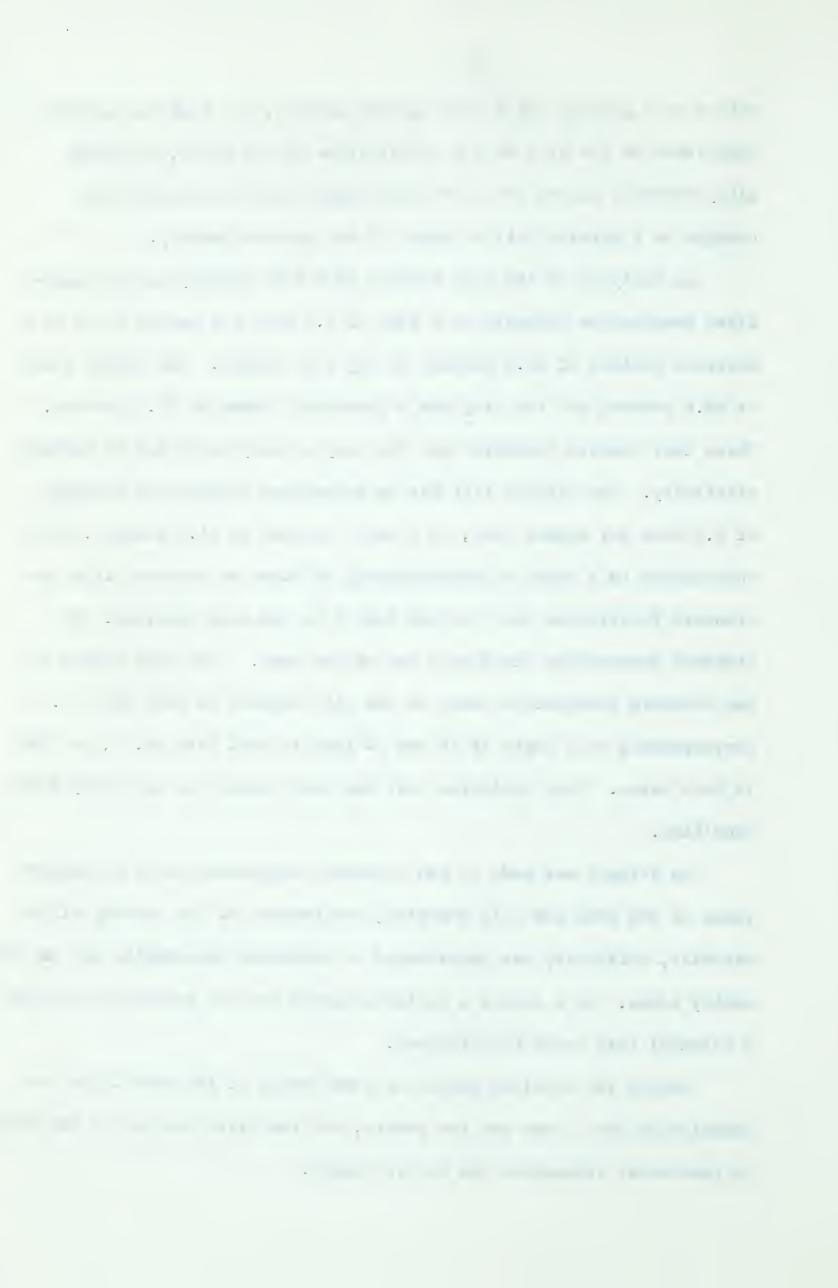
The drill log indicates that the site is overlain by 6 feet of medium plastic, stiff clay. From 6 to 15 feet a layer of medium dense non-plastic glacial till exists, which is in turn underlain by a dense sand layer from 15 to 18 feet. At 18 feet the material changes to a sandy silt, low to non-plastic and contains an occasional pebble and layers of clay till. This material exists to 23 feet below ground surface. No test holes were available near Test Piles 1 and 2 which

extend to a greater depth below ground surface, but from the author's experience on the site during installation of the piles, the dense silty material exists for a few feet deeper and then the material changes to a glacial till of equal if not greater density.

An analysis of the test results show that the clay has an unconfined compressive strength at 5 feet of 3.1 tons per square foot, at a moisture content of 25.0 percent of dry soil weight. The liquid limit is 40.6 percent and the clay has a plasticity index of 21.2 percent. These test results indicate that the clay is very stiff and of medium plasticity. The glacial till has an unconfined compressive strength of 4.4 tons per square foot, at a water content of 11.6 percent. This corresponds to a value of approximately 40 blows as obtained from the Standard Penetration Test in Test Hole 6 on the same material. No Standard Penetration Tests were run on the sand. The blow counts of two Standard Penetration Tests on the silt deposit in Test Hole No. 6 corresponding to a depth of 19 and 22 feet in Test Pile No. 2, was 100 in both cases. This indicates that the silt deposit is in a very dense condition.

An attempt was made to run triaxial compression tests on samples taken of the sand and silt deposits, but because of the density of the material, difficulty was experienced in extruding the samples out of the shelby tubes. As a result a suitable sample was not available on which a triaxial test could be performed.

During the drilling operation some caving of the sand layer was experienced but it was not too severe, and the cross section of the shaft is consistent throughout the entire length.



The subsoil conditions at the location of Test Piles 1 and 2 are very good but they should not be taken as representative for the entire site of The Chemistry and Physics Building. Over some of the other portions of the building site the conditions were not as suitable as in this particular location. The reasons for choosing the area near Test Pile No. 1 were:

- (a) convenience
- (b) accessibility
- (c) pile carrying a small design load. As a result, a pile had to be chosen which best fitted the above conditions even though the subsoil conditions were better than the average.

## (2) SITE OF WEST-END CITY YARDS

Detailed soil conditions together with field and laboratory test results for this site are given in Appendix B, and are summarized on Plate 2B.

The test hole log indicates a layer of gravel from zero to 1 foot, underlain by a firm highly plastic clay down to a depth of 21 feet.

Between 21 and 22 feet a layer of soft clayey silt exists, underlain by a glacial till deposit. The glacial till is soft between 22 and 23 feet, but becomes dense at 25 feet. This deposit is of medium plasticity, grey in color with coal and pea gravel present. None of the test holes were deeper than 30 feet, but it is not possible, on geological grounds, for highly compressible material to exist below the depth investigated.

An analysis of the test results shows that the natural water content is consistent down to a depth of 22 feet, where it increases from

the second secon . A . 4 an average value of approximately 35 to 47.4 percent, and then drops off to 17.6 percent at 25 feet. The natural water content is near the plastic limit throughout the depth investigated with the exception of tests between 20 feet and 22 feet where it approaches the liquid limit, and then falls off to the same value as the plastic limit at 24 feet. When the natural water content in non sensitive clays approaches the plastic limit the soil is in a semi-solid condition, and when it approaches the liquid limit the soil is in a liquid condition.

The liquid limit tests indicate a very highly plastic clay at a depth of 5 feet with the liquid limit varying between 80 and 90 percent. The liquid limit tests on the clay down to a depth of 20 feet resulted in values greater than 50 percent. Because of the increase in silt content at 20 feet the liquid limit drops off to 42.8 percent. The plastic limit in all cases varies between 20 and 30 percent, therefore the plasticity index decreases with an increase in depth.

The Atterberg Limits on the glacial till indicate it to be a medium to highly plastic till, with the plasticity index in the neighborhood of 24 percent.

All the liquid limits on material from this site plotted above the "A" line on the plasticity chart indicating that the material is an inorganic clay. The band of curves formed by plotting the results of tests on the plasticity chart are shown on Plate 3B.

The average of 12 unconfined compressive strength tests on the clay material works out to 1.2 tons per square foot. This indicates that the clay is firm in respect to consistency. A plot of Moisture Content vs Log Compressive Strength of the clay material is shown on

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Plate 4B. No definite trend is evident from this plot.

The average of three unconfined compressive strength tests on the glacial till is 1.7 tons per square foot with a maximum of 2.4 tons per square foot at 26 feet. It is likely that the maximum value would be a typical value for the glacial till since the material above 26 feet is affected by the wet silt deposit at 21 feet. This is brought out in the plot of Moisture Content vs Log Compressive Strength shown on Plate 5B. Even though the number of test results are few, there is a definite trend towards an increase in the unconfined compressive strength with a decrease in moisture content. All the holes made water at a depth of 21 feet below ground surface, because of the wet silty layer present at that depth. Because of this the holes were filled with concrete immediately after the drilling operation had been completed in order to prevent any softening of the material at the base of the pile. The amount of water that seeped in was small, but if the hole had been left open for an hour, probably four or five inches of water would have accumulated in the bottom of a 12 inch diameter hole.

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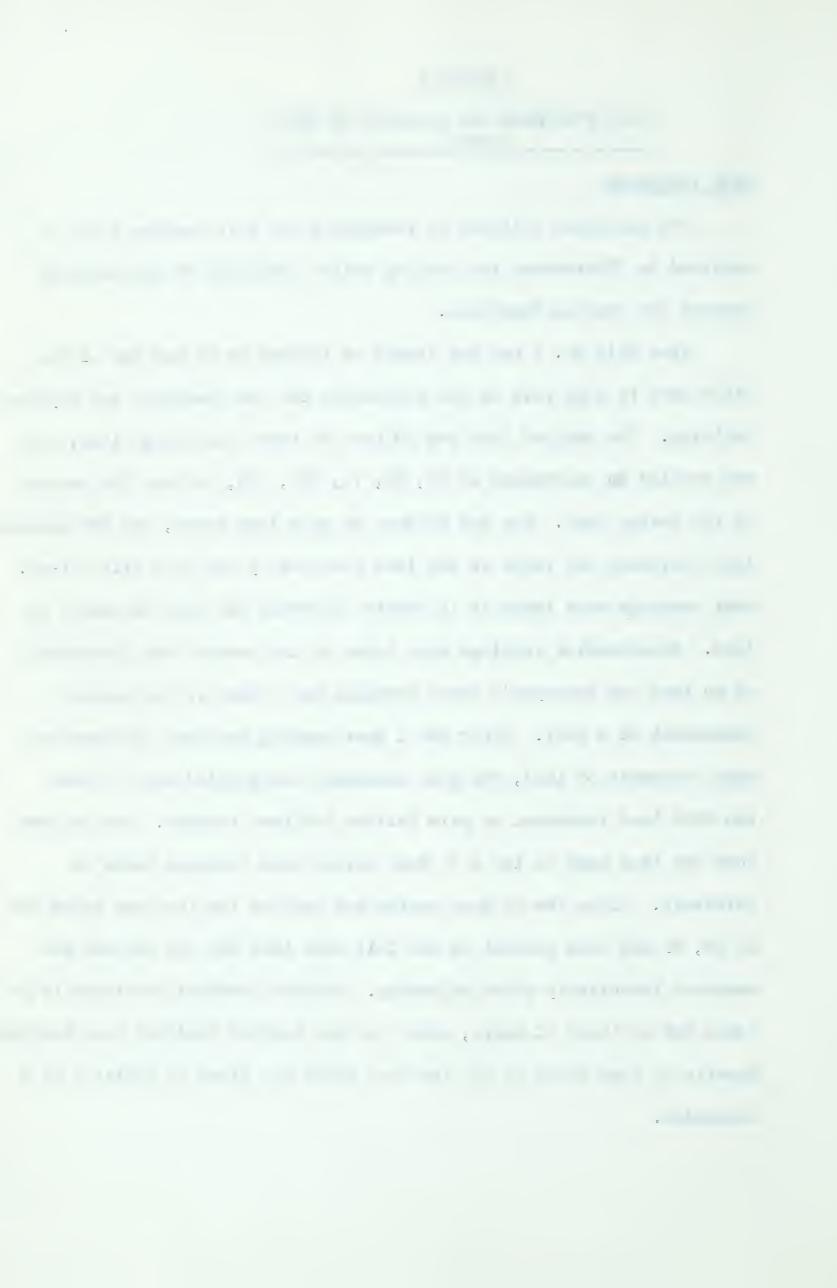
### CHAPTER V

# TEST PROCEDURE AND ANALYSIS OF TEST RESULTS

### TEST PROCEDURE

The procedure followed in conducting the pile loading tests is outlined in "Procedures for Testing Soils" published by the American Society for Testing Materials.

Test Pile No. 1 was not loaded to failure as it was one of the piles used to form part of the foundation for the Chemistry and Physics Building. The applied load was 50 tons or twice the design load, and was applied in increments of 25, 50, 75, 100, 125, 150 and 200 percent of the design load. For the balance of pile load tests, the 200 percent load increment was taken as the load provided by the Test Pile Set-up. Four readings were taken at 15 minute intervals for each increment of load. Extensometer readings were taken to the nearest ten thousandth of an inch and surveyor's level readings were taken to the nearest thousandth of a foot. After the 1 hour reading had been obtained for each increment of load, the next increment was applied until either the 200% load increment or pile failure had been reached. The maximum load was then kept on for a 24 hour period with readings taken at intervals. After the 24 hour period had expired the load was taken off to 50, 25 and zero percent of the full test load and the rebound was measured immediately after unloading. Rebound readings continued to be taken for at least 12 hours, after all the applied load had been removed. Results of load tests on all the Test Piles are given in Tables 2 to 9 inclusive.



### ANALYSIS OF TEST RESULTS

### (1) SITE OF CHEMISTRY AND PHYSICS BUILDING:

Test Pile No. 1 was a 12 inch diameter pile, 23 feet long with a two foot diameter bell and was designed to carry 25 tons.

The load vs Settlement Curve for Test Pile No. 1 is shown on Plate 3A and the results are given in Table 2.

### (a) Pile Behaviour

The total applied load was 50 tons which was 200 percent of the design load. No pile failure occurred using the criteria that pile failure occurs when an increase in load produces a disproportionate increase in settlement (4). Using this criteria, which is put forth in the National Building Code (1953), the allowable design load permitted on this pile is greater than 25 tons. The design load permitted should in no case exceed the load which produces a net settlement of 0.01 inches per ton of gross load applied. The net settlement at 50 tons of applied load was 0.06 inches and the allowable settlement permitted by the Code at an applied load of 50 tons is 0.50 inches. Therefore on the basis of either criteria no pile failure occurred and it would appear that the pile has a factor of safety greater than 3.

### (b) Theoretical Analysis

The pile is embedded from zero to 6 feet in clay, from 6 to 15 feet in glacial till, from 15 to 18 feet in dense sand and from 18 to 23 feet in a sandy silt. The shear strength of the clay is 1.5 tons per square foot, and that of the glacial till is 2.2 tons per square foot. For the clay and the glacial till the shearing resistance is taken as one-half the unconfined compressive strength, but for the case of the granular

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material from 15 to 23 feet it is difficult to compute the shearing resistance of the soil along the sides of the pile. However, assuming that the shearing resistance for the granular material along the sides of the pile can be determined from the following equation:  $s = p \tan \phi$  (5), where s = shear strength of the material, p = effective or intergranular pressure,  $tan \phi = coefficient$  of internal friction,  $\phi = angle of internal friction$ .

The angle of internal friction for a dense uniform sand that consists primarily of rounded grains is approximately 35°. Assuming an average angle of internal friction in this case for the sand and silt material between 15 and 23 feet as equal to 25°, and an average effective pressure of 1 ton per square foot, the average shear strength of this material works out to 0.47 tons per square foot. The total ultimate value for skin friction on this pile would be the sum of the skin friction of each of the materials penetrated. This works out to 28 tons for the 6 feet of clay penetrated, 62 tons for the 9 feet of glacial till penetrated, and 12 tons for the 8 feet of sand and silt penetrated, making a total of 102 tons for skin friction.

To determine the end-bearing capacity of a two foot diameter bell the formula  $qdr = \forall D_f N_q + 0.6 \forall r N_g$  (5), was used. In this formula  $qdr = 0.6 \forall r N_g$  (5), was used. In this formula  $qdr = 0.6 \forall r N_g$  density of the ultimate bearing capacity of a circular footing.  $\forall r = 0.6 \forall r = 0.$ 

Using a value of  $\phi$  = 25° in determining the bearing capacity



factors, and assuming that the density of the soil at 23 feet is 130 lbs. per cubic foot, then the ultimate bearing capacity of a two diameter bell is 20 tons per square foot. Therefore the ultimate capacity of this pile based on end-bearing only should be 62 tons. Combining the calculated ultimate capacity for skin friction and end-bearing the calculated ultimate capacity of this pile should be 164 tons.

In order for peak values to be used as calculated above the stress strain characteristics must be the same for all the material through which the pile has penetrated. It is extremely unlikely that it would be and therefore peak values would not be fully developed. Therefore the maximum calculated capacity of the pile would not likely be the true maximum. However, the theoretical analysis does tend to confirm the conclusion based on the result of the load test that the pile probably has a factor of safety somewhat in excess of 3.

Test Pile No. 2 was a 12 inch diameter pile 23 feet long with no bell and was constructed to be loaded to failure.

The Load vs Settlement Curve is shown on Plate 3A, and the results are given in Table 3.

### (a) Pile Behaviour

No pile failure occurred at the total applied load of 70 tons, based on criteria previously mentioned. The net allowable settlement under an applied load of 70 tons is 0.70 inches and the net settlement recorded under the applied load was 0.2729 inches. As a result there must be a considerable effect due to skin friction.

The total settlement recorded was 0.3617 inches of which 28 per

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cent or 0.0988 inches is due to elastic deformation.

### (b) Theoretical analysis

The pile penetrates the same material as in the case of Test Pile No. 1, and since it is also the same diameter, the ultimate calculated value for skin friction of 102 tons must also apply to this pile. The base of the pile in this case is 1 foot in diameter and the ultimate capacity based only on end-bearing should be 16 tons. Then theoretically the failure load of this pile should be 118 tons. Here again, the calculated ultimate values would have to be considered as peak values since the stress strain characteristics for the soils penetrate are not the same.

The Load vs Settlement Curves indicate that Test Pile No. 2 settled 35 percent more than Test Pile No. 1 at an applied load of 50 tons. This is due to the smaller bearing area of Pile No. 2. However most of the difference in settlement took place for the loads up to 25 tons, and beyond that the curves are parallel. It thus appears that the only function of the larger bearing area in this case is to reduce the total settlement of the pile. The difference in total settlement of the two piles at the design load of Test Pile No. 1 is 0.05 inches. Considering that at 25 tons applied load the total settlement of Test Pile No. 2 was 0.085 inches, it would appear that for the type of structure proposed, a 12 inch diameter friction pile 23 feet deep would have been suitable in this case.

### (II) SITE OF WEST-END CITY YARDS

Since Test Pile No. 3 is a combination of end-bearing and skin friction the test results will not be discussed until the test results

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of Test Piles 4 and 5 have been analysed.

Test Pile No. 4 was a 12 inch diameter pile, 23 feet long, with no bell and was suspended 3 feet above the bottom of the hole. This pile was constructed to determine the effect of skin friction.

Two tests were performed on the pile and the Load vs Settlement Curves are shown on Plate 6B, and the Test Results are given in Tables 4 and 5. In Trial No. 1, the pile was loaded to 37.5 tons and then as there was insufficient load on the test pile setup to apply the next increment of load, the pile was unloaded for 24 hours. The test was then resumed and the pile loaded to failure. This possibly is the explanation for the plot of the Load vs Settlement Curve for Trial No. 1 not forming a smooth curve.

### (a) Pile Behaviour

Pile failure occurred at an applied load of 65 tons based on the criteria of disproportionate settlement, and therefore an allowable design load of 30 tons would be permitted on this pile. The net settlement at 30 tons applied load was approximately 0.08 inches while on the bases of the second criteria a net settlement of 0.30 inches would be permitted. The net settlement of this pile after 24 hours under an applied load of 65 tons was 2.146 inches for Trial No. 1 and 2.164 inches for Trial No. 2.

The elastic deformation in Trial No. was 2.3 percent or 0.05 inches.

In Trial No. 2 the elastic deformation was 3.6 percent or 0.08 inches.

### (b) Theoretical Analysis

The highly plastic clay material on this site has a Liquid Limit varying between 52 and 89 percent and the average moisture content is

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approximately 36 percent. Therefore the clay should behave as a saturated cohesive material.

The pile is embedded in 1 foot of gravelly material which is underlain by a highly plastic clay layer from 1 to 21 feet below ground surface. Below this exists a 1 foot thick stratum of sandy silt which is in turn underlain from 22 to 26 feet by glacial till. The clay is stiff and on the basis of 12 test results has an average shear strength of 1200 lbs. per square foot. No tests were run on the sandy silt layer which is low to medium plastic and has a very soft consistency. The unconfined compression strength of the glacial till increases from 1.2 tons per square foot at 23 feet to 2.4 tons per square foot at 26 feet. The top of the glacial till has softened up due to the presence of the wet silt layer above it. Based on an average of three tests the shearing strength of the glacial till is 1600 lbs. per square foot. The ultimate capacity of this pile should then be the sum of the skin friction for the length of pile penetrating the clay, the length penetrating the silt and the length penetrating the glacial till. It is assumed that the shear strength of the gravel fill is equal to the average of the clay. It will be further assumed that the shear strength of the silt deposit is equal to 0.12 tons per square foot (13). Also, since the pile penetrates only into the soft glacial till, therefore in calculating the skin friction for that portion of pile, a shearing strength equal to 1200 lbs. per square foot will be used. This is based on the sample tested at 23 feet. Using the above values, the average shearing resistance of the soil is 1160 lbs. per square foot, and the theoretical ultimate capacity of the pile works out to 42 tons. This is considerably less than the load of



of tons which was required to fail this pile. Since we are dealing with a saturated non-sensitive clay soil the calculated ultimate capacity should not vary greatly from the ultimate capacity as determined by means of a pile load test. The difference between the two results is considerable in this case, therefore possibly the top 3 feet of the pile which penetrated frozen soil affected the test results.

In arriving at the actual value for the shear strength along the length of the pile based on the failure load, the weight of the pile should be added to the applied load of 65 tons. The weight of a column of concrete 12 inches in diameter and 23 feet long is 1.35 tons, therefore the failure load should be taken as 66 tons. On this basis the average ultimate shear strength of the soil works out to 1800 lbs. per square foot for a pile 23 feet long. This is 640 lbs. per foot higher than the value for the shear strength of the soil based on laboratory test results. Assuming that the top 3 feet of frozen soil accounted for the difference of 24 tons between the calculated and the actual ultimate capacity of the pile, then the tangential adfreezing (6) strength of this material works out to 5,000 lbs. per square foot.

The tangential adfreezing strength is defined as the resistance to the force that is required to shear off an object which is frozen to the ground, and to overcome the friction along the plane of its contact with the ground. The value of the tangential adfreezing strength varies with many factors (20). In comparison the value given for a clay soil at a temperature of 4 120F, and at a moisture content of 34.6 percent

is 24.7 tons per square foot. Assuming that the full value of the tangential adfreezing effect is developed in the top 3 feet of frozen soil, then an applied load of 242 tons would be required to overcome this effect. Since pile failure did occur, and the theoretical analysis does not agree with the actual field test, it would have to be assumed that the freezing-in of the top 3 feet had some effect on the test results, but to a considerably smaller extent than indicated by the above analysis.

Two tests were run on this pile in an attempt to determine the effect of remolding of the clay after shear failure had occurred and also to try and determine the effect of freezing-in of the pile on the ultimate bearing capacity. The test results did not vary sufficiently to draw any conclusions of the above effect on the ultimate capacity.

Test Pile No. 5 was a 12 inch diameter pile, 26 feet long, and flared out to 16 inches in diameter at the bottom end (Photo 1). The height of the flare was 8 inches. Test Pile No. 5 was bearing on glacial till and was constructed to determine the effect of end bearing in a pile foundation.

Load vs Settlement Curves for the three tests performed on this pile are shown on Plate 7B, and the results of the three tests made on this pile are given in Tables 6, 7 and 8.

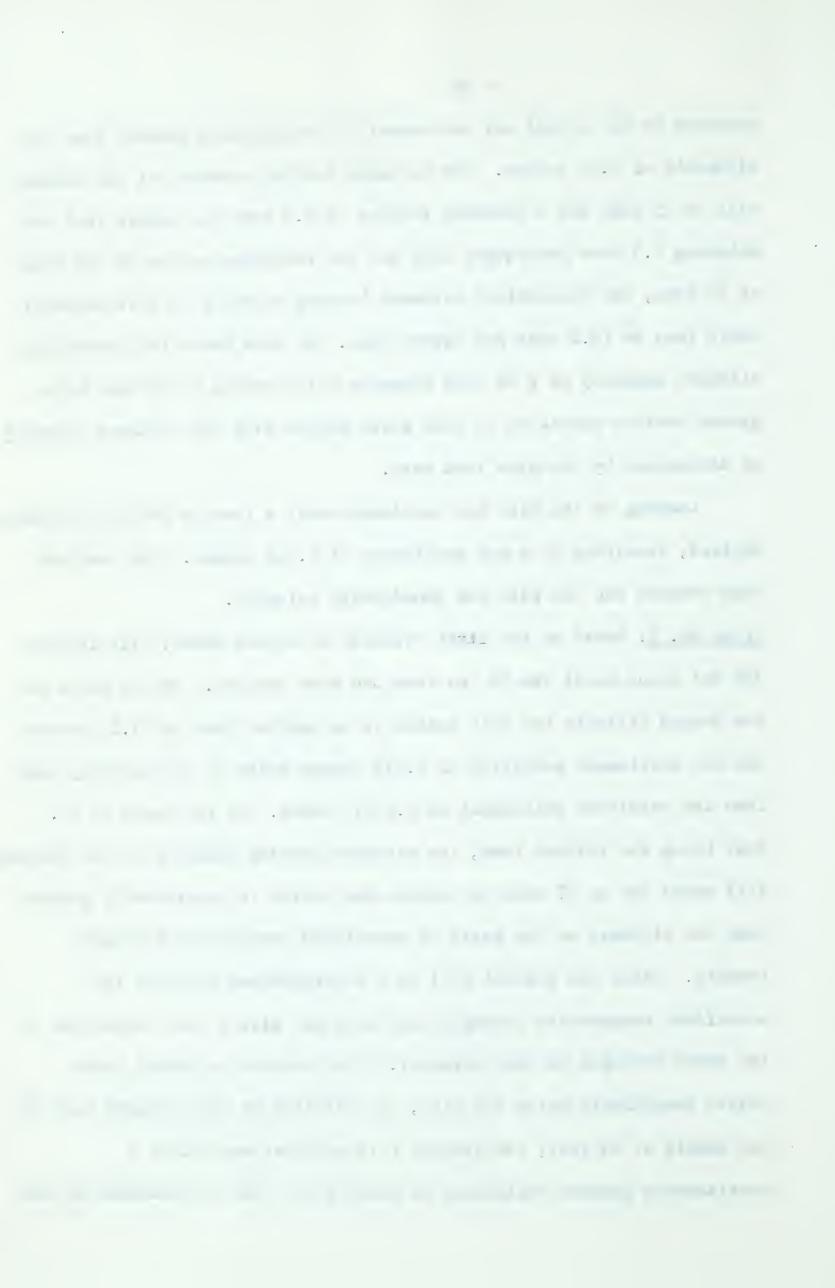
# (a) Pile Behaviour and Theoretical Analysis

Trial No. 1: based on the criteria of disproportionate settlement, pile failure occurred with the application of the first test load increment of 12.5 tons. The net settlement under this load was 1.56 inches therefore on the basis of the second criteria pile failure also

in the second se  occurred as the actual net settlement is considerably greater than the allowable of 0.12 inches. The ultimate bearing capacity of the glacial till at 25 feet for a circular footing is 8.9 tons per square foot and allowing 1.3 tons per square foot for the confining action of the soil at 26 feet, the theoretical ultimate bearing capacity of this material would then be 10.2 tons per square foot. On this basis the theoretical ultimate capacity of a 16 inch diameter pile bearing at 26 feet below ground surface should be 14 tons which agrees with the ultimate capacity as determined by the pile load test.

Loading of the pile was continued until a load of 50 tons had been applied, resulting in a net settlement of 7.416 inches. The load was then removed and the pile was immediately reloaded.

Trial No. 2: based on the first criteria as stated above, pile failure did not occur until the 50 ton load had been applied. On the basis of the second criteria the pile failed at an applied load of 37.5 tons as the net settlement permitted is 0.375 inches which is considerably less than the resultant settlement of 0.6357 inches. On the basis of 37.5 tons being the failure load, the ultimate bearing capacity of the glacial till works out to 27 tons per square foot which is considerably greater than the ultimate on the basis of unconfined compressive strength results. Since the glacial till is a heterogeneous material the unconfined compressive strength test does not give a true indication of the shear strength of this material. Also because no denser layer exists immediately below the pile, as indicated by the strength test on the sample at 30 feet, the glacial till material must offer a considerable greater resistance to penetration than is indicated by the



unconfined compressive strength test. A better indication of the penetration resistance of the glacial till would have been obtained from a standard penetration test, but as no equipment was available to run this test, it was decided to run unconfined compression tests instead.

The net settlement of the pile 24 hours after the 50 ton load was applied was 6.35 inches, and thus the total settlement for the first two tests was 13.76 inches. The elastic deformation was 2.7% or 0.1738 inches.

Trial No. 3: based on either of the two criteria for pile failure, no pile failure occurred at an applied load of 55 tons. The net settlement 24 hours after the 55 ton load was applied was 0.384 inches as compared to the allowable of 0.55 inches. The elastic deformation was 47.6 percent or 0.348 inches. Even though possibly some of the load is being taken by skin friction on the flared out portion of the pile, it is difficult to make any analysis of this, and since the flared out portion is only 8 inches high, the amount of skin friction would be small in comparison to the end-bearing. Assuming that penetration is being resisted by end-bearing alone then at 55 tons applied load, the bearing capacity being developed is 40 tons per square foot. It would appear that on the basis of the three test results, the resistance of the glacial till to penetration increases considerably with a relatively small penetration of the test pile.

On the basis of the three tests performed on this pile it would have to be assumed that the unconfined compression strength test results together with the necessary adjustment for the depth factor give

accurate results for the end-bearing capacity of a drilled cast-inplace pile. In the case of a pile being driven into the glacial till
and therefore causing displacement of the material, the standard
penetration test would be more suitable for determining the design
load.

Test Pile No. 3 was a 12 inch diameter pile, 26 feet long with no bell, and was constructed to be loaded to failure.

The Load vs Settlement Curve is shown on Plate 8B, and the test results are given in Table 9.

### (a) Pile Behaviour

No pile failure occurred at the applied load of 70 tons. This is in accordance with the criteria of disproportionate settlement.

Also, on the basis of the second criteria no pile failure occurred as the allowable net settlement permitted is 0.70 inches and the actual net settlement that occurred was 0.2221 inches. The elastic deformation was 25.8 percent or 0.0774 inches.

### (b) Theoretical Analysis

Theoretically this pile derives its carrying capacity from a combination of skin friction and end-bearing, as compared to the simple case of a friction pile in Test Pile No. 4 and an end-bearing pile in Test Pile No. 5. The ultimate theoretical value for skin friction is the sum of the skin friction through 21 feet of clay, one foot of silt and 4 feet of glacial till. It will be assumed that the 1 foot of fill to have a shear strength equal to the average shear strength of the clay. The theoretical ultimate value for skin friction through the fill and the clay, based on an average shear strength of 1200 lbs. per square foot

is 40 tons. Here again assuming that the one foot silt layer has an average shear strength of 0.12 tons per square foot, then the theoretical ultimate value for skin friction through this layer is 0.4 tons. The theoretical ultimate value for skin friction through the glacial till, based on an average shear strength of 1600 lbs. per square foot is 10 tons. Therefore the theoretical ultimate value for skin friction on this pile is 50 tons. Allowing 1.3 tons per square foot for the depth factor then, the theoretical ultimate end-bearing capacity of a 12 inch diameter pile bearing on the glacial till at 26 feet is 8 tons. Combining the ultimate values for skin friction and end-bearing, the pile should theoretically have an ultimate capacity of 58 tons, assuming the clay and the glacial till to have the same stress strain characteristics.

Since the pile was not tested to failure and, as in the case of Test Pile No. 4, freezing-in of the pile did occur, it is difficult to draw any conclusions from the theoretical analysis based on the field test results. As in the case of the friction pile it is likely that the freezing-in of the pile affected the test results to a certain extent:

For comparison purposes a plot of Load vs Settlement Curves for all the Test Piles with the exception of Test Pile No. 5 is shown on Plate 7B. In comparison of 1 hour readings the least settlement was recorded in Test Pile No. 1, up to a test load of 36 tons. For test loads between 36 and 70 tons, Test Pile No. 3 recorded the least settlement. The Load vs Settlement Curve for Test Pile No. 2 is below that for Test Pile No. 3 and the difference increases with an increase

in load. The difference at 70 tons was 17 percent while at 50 tons it was 14 percent. This does not agree on the basis of strength test results for the two types of soils but does agree as far as the theory of consolidation for rate of settlement is concerned. In the case of Test Pile No. 3 because of impervious nature of the clay as compared to the sand, consolidation does not take place until a considerable time after the load is applied. In comparing the load settlement curves for Test Piles No. 1 and 3, the increased end area in Test Pile No. 1 had no effect in reducing the settlement for loads greater than 36 tons.

Curves for Test Pile No. 4 are below those of Test Piles 1, 2 and 3, with the exception of that between 21 and 45 tons, where the curve for Trial No. 2 is above that of Test Pile No. 2. This is possibly due to remolding effect of the clay soil.

Based on 24 hour readings the settlement of Test Pile No. 2 was 0.1237 inches while Test Pile No. 3 settled 0.216 inches, which is in agreement with the theory of consolidation.

A plot of stress vs strain based on field test results for the skin friction and end-bearing piles is shown on Plate 10B. The modulus of elasticity for the soil along the sides of the pile is greater than for the soil at the bottom of the pile. The plot of the two Trials for the friction pile are in close agreement. In the case of the end-bearing pile, Trial No. 3 definitely has a higher modulus of elasticity than Trial No. 2. This plot indicates that the material along the sides of the pile is stiffer than the material at the base of the pile. As a result in the case of the combined skin friction and end-bearing pile, the skin friction component would carry the greater percentage of the applied load.

A summary of pile load test results is given in the following table:

TABLE A - SUMMARY OF TEST RESULTS

		PILE NO.			
ITEM	1	2	3	4	5
Applied Load (tons)	50	70	70	65	55
Measured Ultimate Load (tons)	00	-	60	65	12.5
Ultimate Load Based on shear strength of the soil (tons)	164	118 (-)	58	42	14

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### CHAPTER VI

### CONCLUSIONS AND RECOMMENDATIONS

### **CONCLUSIONS:**

The following conclusions can be drawn from the results obtained from the testing program.

### (A) Site of Chemistry and Physics Building.

- (1) Piles tested did not fail under test load. It would appear that the pile tested forming part of the foundation for the Chemistry and Physics Building has a factor of safety somewhat greater than 3. This was expected as the laboratory strength test results indicated that the sub surface conditions at this particular location were better than at other locations on this site.
- (2) Since neither of the piles was loaded to failure, it is difficult to draw any conclusions on the accuracy of the theoretical analysis.

### (B) West-End City Yards.

- (3) Test Results for the friction pile did not check with theoretical analysis and the most reasonable explanation appears to be due to the fact that the top 3 feet of pile was in frozen soil.
- (4) Test results for the end-bearing pile check with the theoretical analysis.
- (5) Test results for the combined skin friction and endbearing pile do not check with theoretical analysis, and here again the discrepancy appears to be due to the effect of the frozen soil.

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### **RECOMMENDATIONS:**

For possible future research into the carrying capacity of castin-place piles the following points are put forward:

- (1) The test set up was satisfactory for the tests
  performed. In any future testing it may be found that the testing could
  be performed more economically by the use of anchor piles.
- (2) For better anchorage of the reference points, a hole down to a depth of about 10 feet, filled with concrete could be used for short term tests, and down to at least 20 feet for long term tests.
- (3) A nest of springs may be used on the top of pile in order to better maintain each increment of load especially when a constant load is to be maintained for any length of time.
- bearing capacity of cast-in-place concrete piles with time. It had been hoped that conclusions could be drawn on the amount of skin friction that could be considered in the design of cast-in-place pile foundations, but since the modulus of deformation was the same for both types of soils therefore no conclusions can be drawn. It is hoped though, that enough data has been presented to stimulate interest in cast-in-place piles sufficiently to warrant more research in order to establish a uniform design criteria for foundations of this type.

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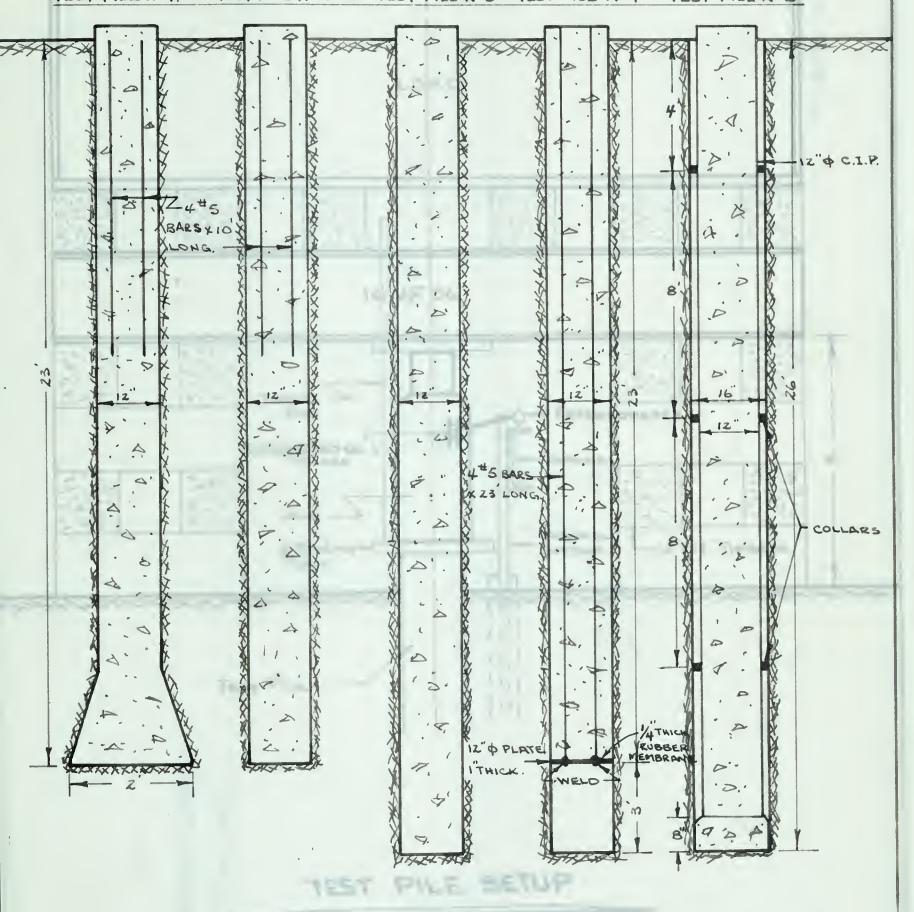
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TEST PILE NO 1 .- TEST PILE Nº 2 - TEST PILE Nº 3 -TEST PILE Nº 4 - TEST PILE Nº 5

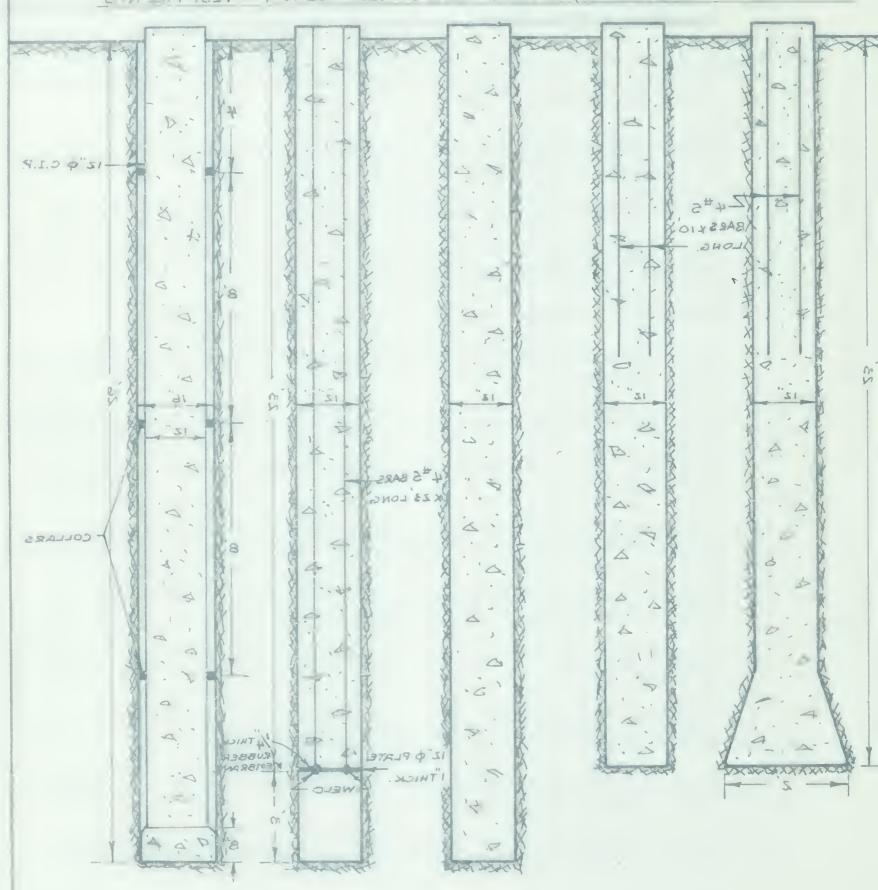


TYPICAL SECTIONS OF PILES TESTED

SCALES - HORIZONTAL 1"= 2" VERTICAL 1"= 4".

FEBRUILRY 26, 1959.

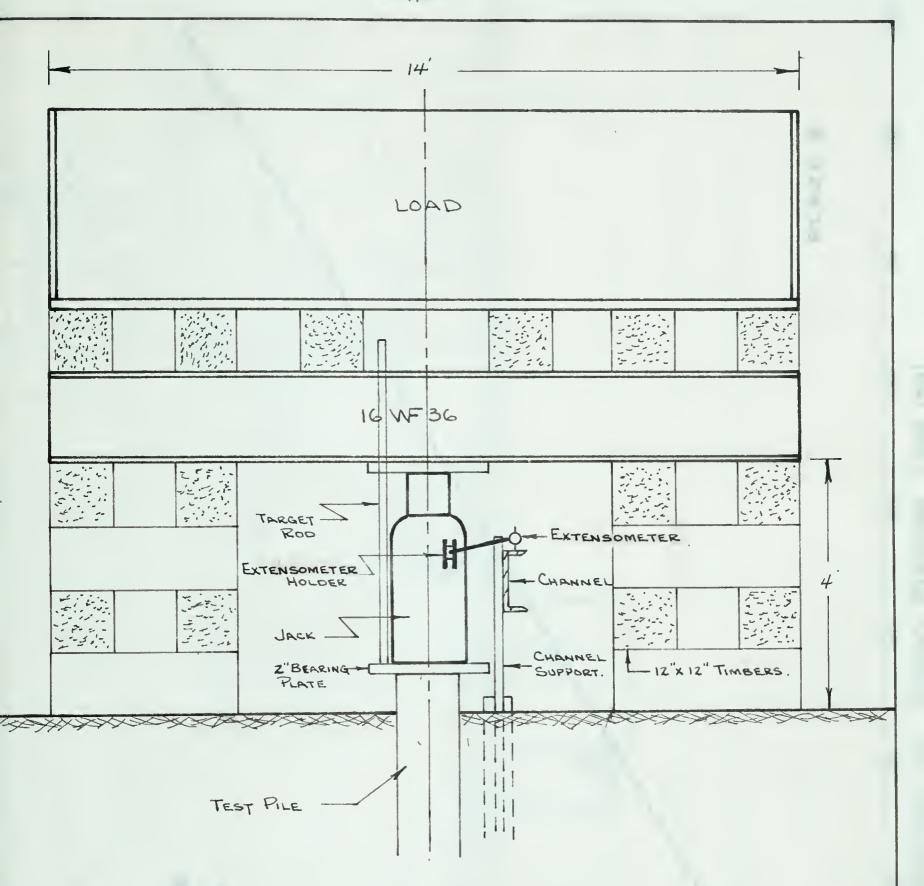
PLATE I.



TYPICAL SECTIONS OF PILES TESTED

SCALES - HORIZONTAL 1'= 2'
VERTICAL 1": 4'.

FEBRUIL Y IL 1957.



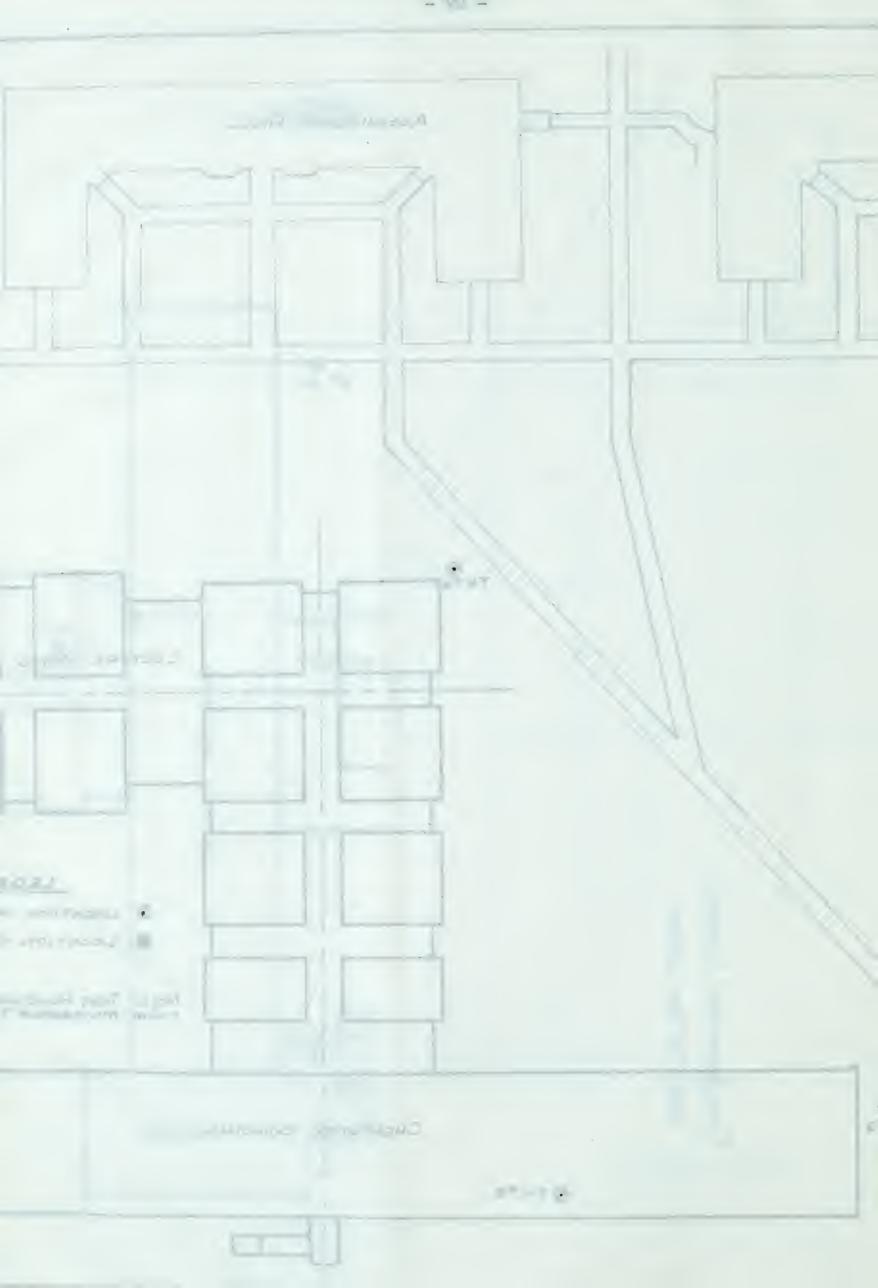
# TEST PILE SETUP

NOT TO SCALE

FEB. 19, 1959.

PLATE 2.

SETWAY and to be 84/9 THAT! PILE SETUP TEST hand or pett ASP B AND 5 374.19



# SOIL PROFILE & SUMMARY of LABORATORY RESULTS.

PROJECT :- TEST PILE Nº 2.

SITE: PHYSICS & CHEMISTRY BUILDING! UNIVERSITY of ALBERTA CAMPUS, EDMONTON, ALTA.

PLATE ZA.

ATTERBERG LIMITS.	DEPTH (ft) SAMPLES	BoL	DESCRIPTION	PENETRATION	UNCONFINED COMPRESSION TEST		
10 20 30 40 50	SAMPLES	3	. 3	RESISTANCE	w/c %	U	
10 20 30 40 50	2		CLAY:- light brown, stiff, moist, medium plastic, rust lenses present.				
	S-1				25.8	3.1	
	10 S-Z	4	GLACIALTILL: pockets of coal present, weathered, pebbles present, moist, non- plastic, medium dense.	38 blows	11.to	4.4	
	14	Q 6.		#Z blows	10.0		
	16		SAND: clean, dense, light brown, fine to medium graded dry, siltier with depth.				
	20 5-4	A	SILT: moist, sandy, low to non-plastic, occasional pebble, layers of clay till present.	loo blows	10.8		
	22.	<i>I</i>	END OF HOLE -23' - BOTTOM OF PI				

### ~ LEGEND ~

• & W/C - WATER CONTENT (PERCENT of DRY SOIL WEIGHT) A - LIQUID LIMIT.

U- UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ.FT)

(S- SHELBY TUBE SAMPLES.

O-Plastic LIMIT.













COMBINATIONS OF ABOVE SHOWN WITH PREDOMINATE SOIL TYPE IN HEAVY LINE & MODIFYING IN LIGHT LINE. NOTE: - ABOVE LOG ALSO APPLIES TO TEST PILE Nº 1.

> \* - STANDARD PENETRATION TEST RESULTS - TAKEN FROM TEST HOLE NO. 6 OF SOILS REPORT AS PREPARED BY MATERIALS TESTING LABORATORIES ON ABOVE SITE.

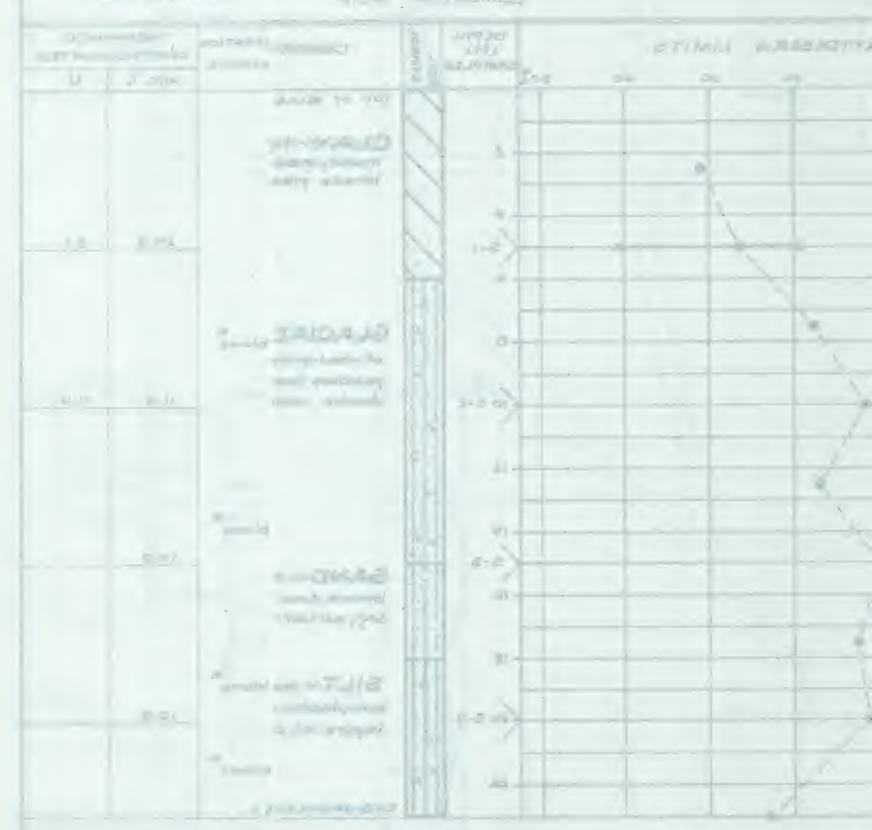
### PRESSURE TRAFF HALL 18" A.

SOIL PROFILE I SUMMARKS I SUPPLY

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description of the second And - many

PLATE RA



## ~ CURSUS.4 ~

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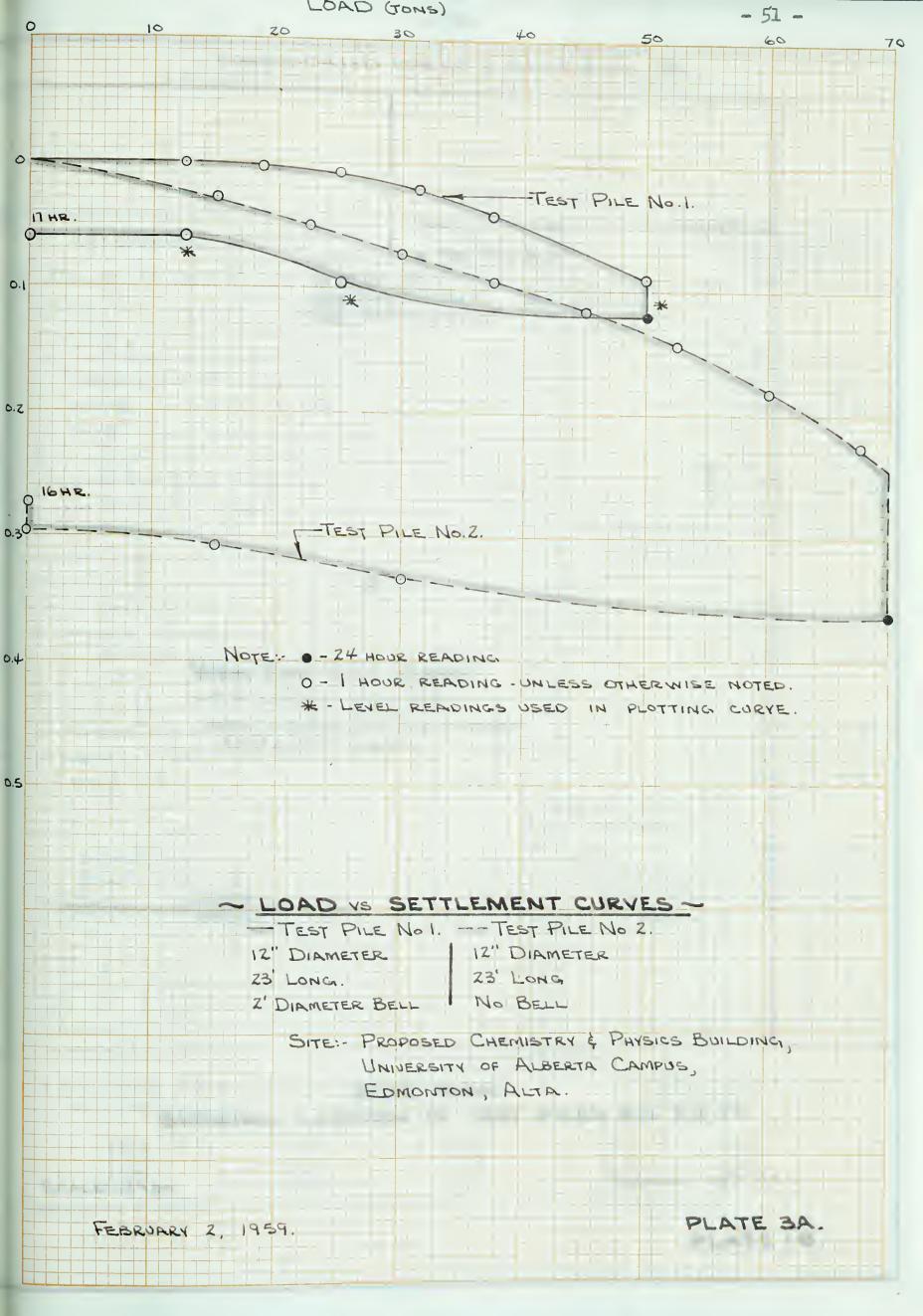






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H.

OFFICE NO. 1

OFFICE NO. 2

OFFICE NO. 2.

NOTE: 0-24 HOUR EASING

13.

0 - I HOUR READING - JULE'S SHER WISE NOTED & - LEVEL READINGS USED I PLOTTING CURYE.

## - LOAD VE SETTLEMENT CURVES -

TEST PILE II. -- I = FILE II. 0 ?

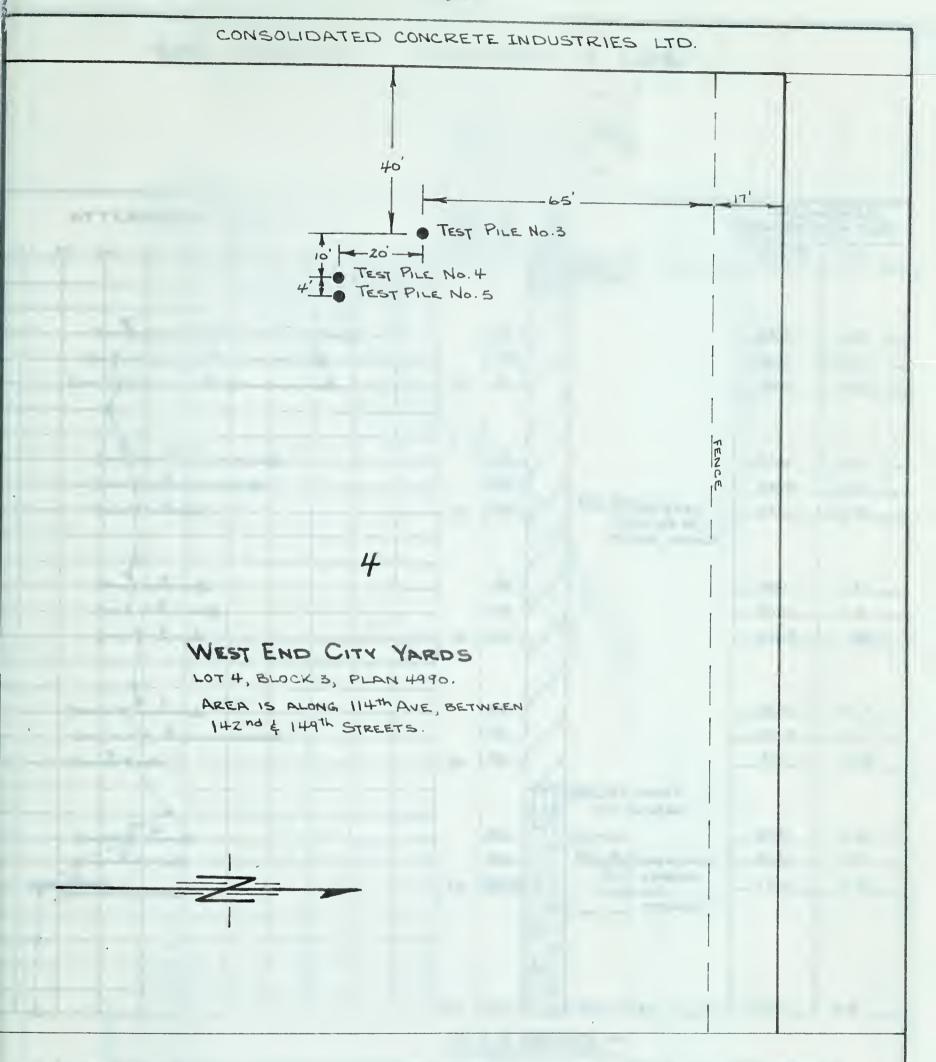
12" DI METE

2" LONG.

2" DIAMETER B LL

2" DIAMETER B LL

SITES - PROPORTO STANDET DINCHINA DINCH



SITE PLAN
SHOWING LOCATION OF TEST PILES No's 3,4, \$5

SCALE - 1"=30"

FEBRUARY 25th, 1959.

PLATE IB.

CONVINCENTIAL OF SERVICE OF THE DESCRIPTION OF THE OWNER OWNER OF THE OWNER OW

4

MAJE STIE CHARLE LICENTIAN DE TEST PRINCIPAL SAMMONE

INFO PER NON SOUTH

Service Lucione

### SOIL PROFILE & SUMMARY of LABORATORY TESTS

PROJECT:- TEST PILES Nº 3, 4, \$ 5.

SITE: WEST END CITY YARD,

114th Ave & 144th STREET,

EDMONTON, ALTA.

ATTERBERG LIMITS								E.)	PLES	SOIL	DESCRIPTION	COMPRESSION TES		
							90.0	<b>.</b>	CFPT.	SAMPL	50 m	TOP OF PILES 8445.	w/c%	U
20	30	-	30	6.0				1072		"		FILL:- gravel, clay.		
			4							14			35.7	1.2
			5							15	\ /		32.6	1-1
0	7)-		3			-			- 5	13	/ /		32.6	2.0
		4 5 3	<b>A</b>		A				- 10	24 25 23		CLAY:- highly plastic, mottled grey and brown, high salt content at B', nugget structure to 12', firm, moist.	37.6 38.3 39.6	1.1
		4 5 3	A						- 15	34 35 33			36.6 36.8	1.2 1.3 1.5
 		4	4		<u> </u>					44			36.4 40.Z	9.9 0.7
0	3								- 2°	43			37.1	9.0
•	-5	4	<b>A</b>							54 5s	44	SILT: soft, grey, clayey, very moist. low to medium plastic, making water BOTTOM OF PILE 4: GLACIAL TILL: soft at 22' becomes	27.8 26.6	1.2
3	*								. 25	5 <sub>3</sub> -64	1111	dense at 25', grey, medium plastic, coal, pea gravel and occasional cobble present.	/7.6	z.4
											4 - A - X			
			L			1	1		30	63		END OF HOLE -30 - TEST PILE Nº 3.	140	2.5

#### ~ LEGEND ~

• & W/C - WATER CONTENT (PERCENT of DRY SOIL WEIGHT)

A - LIQUID LIMIT.

U - Unconfined Compressive Strength (Tons/SQ.FT.)
14 - INDICATES - SAMPLE Nº 1, TEST HOLE Nº 4.













COMBINATIONS OF ABOVE SHOWN WITH PREDOMINATE SOIL TYPE IN HEAVY LINE AND MODIFYING IN LIGHT LINE.

# SOIL PROPILE & SUMMARRY II LEBORAT

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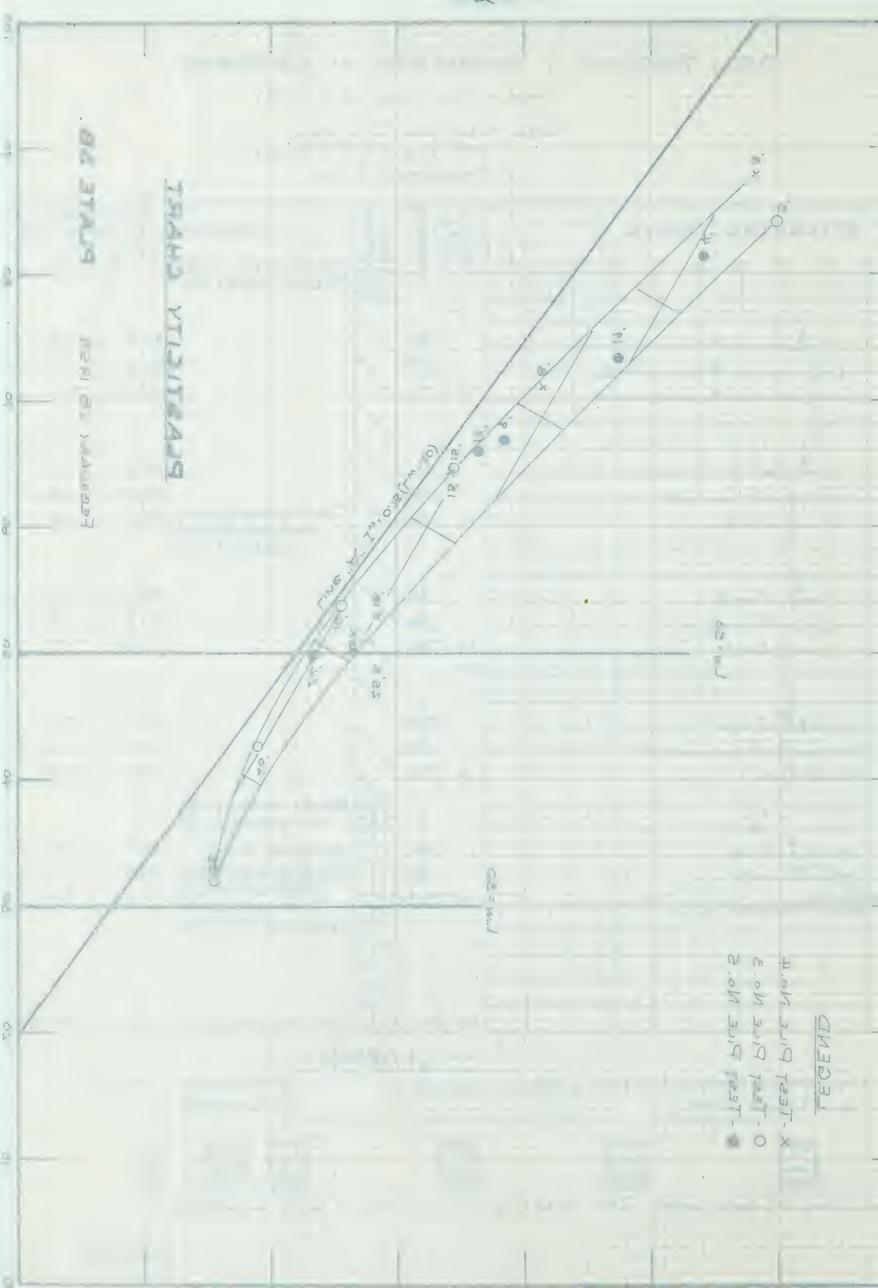
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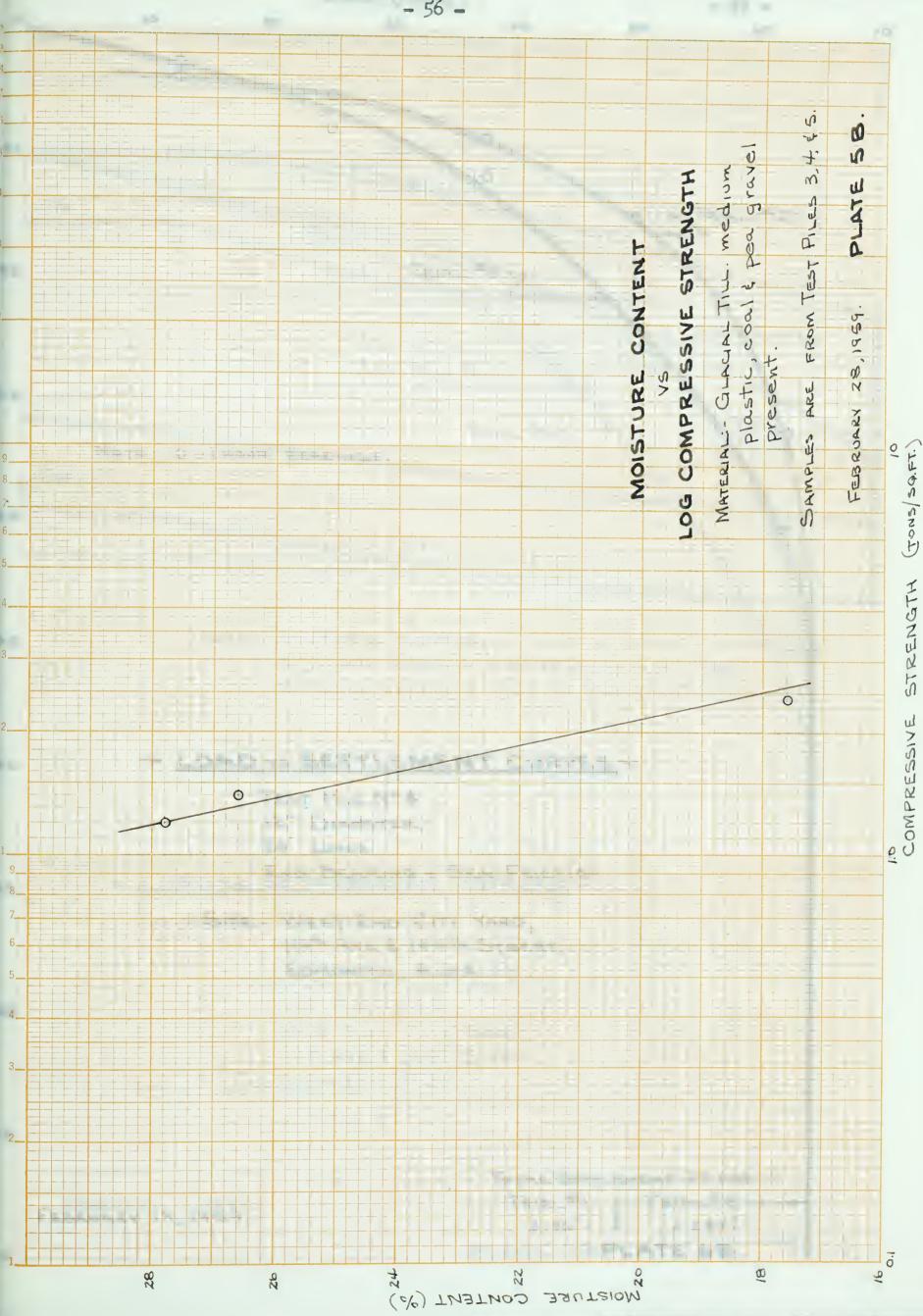




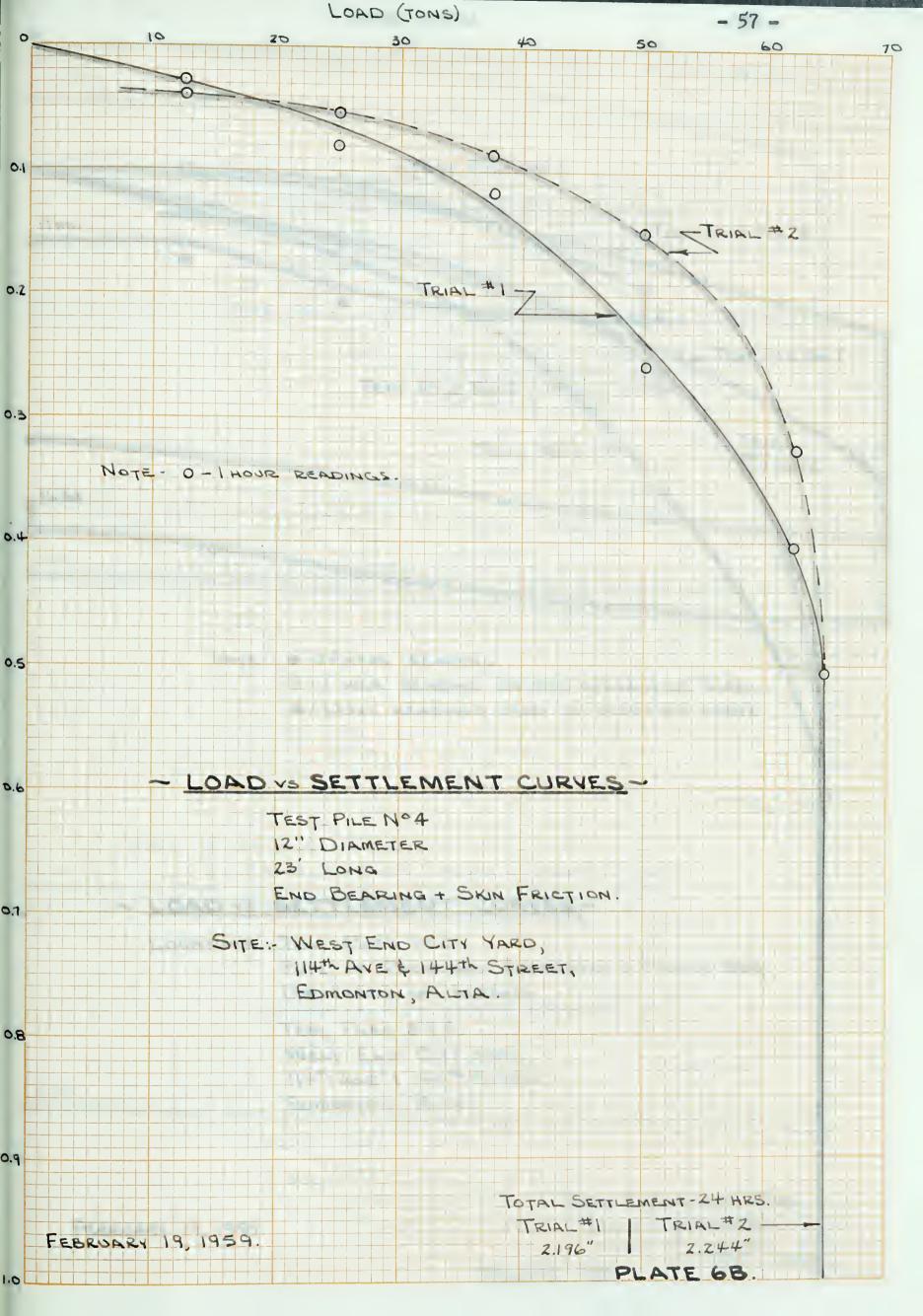


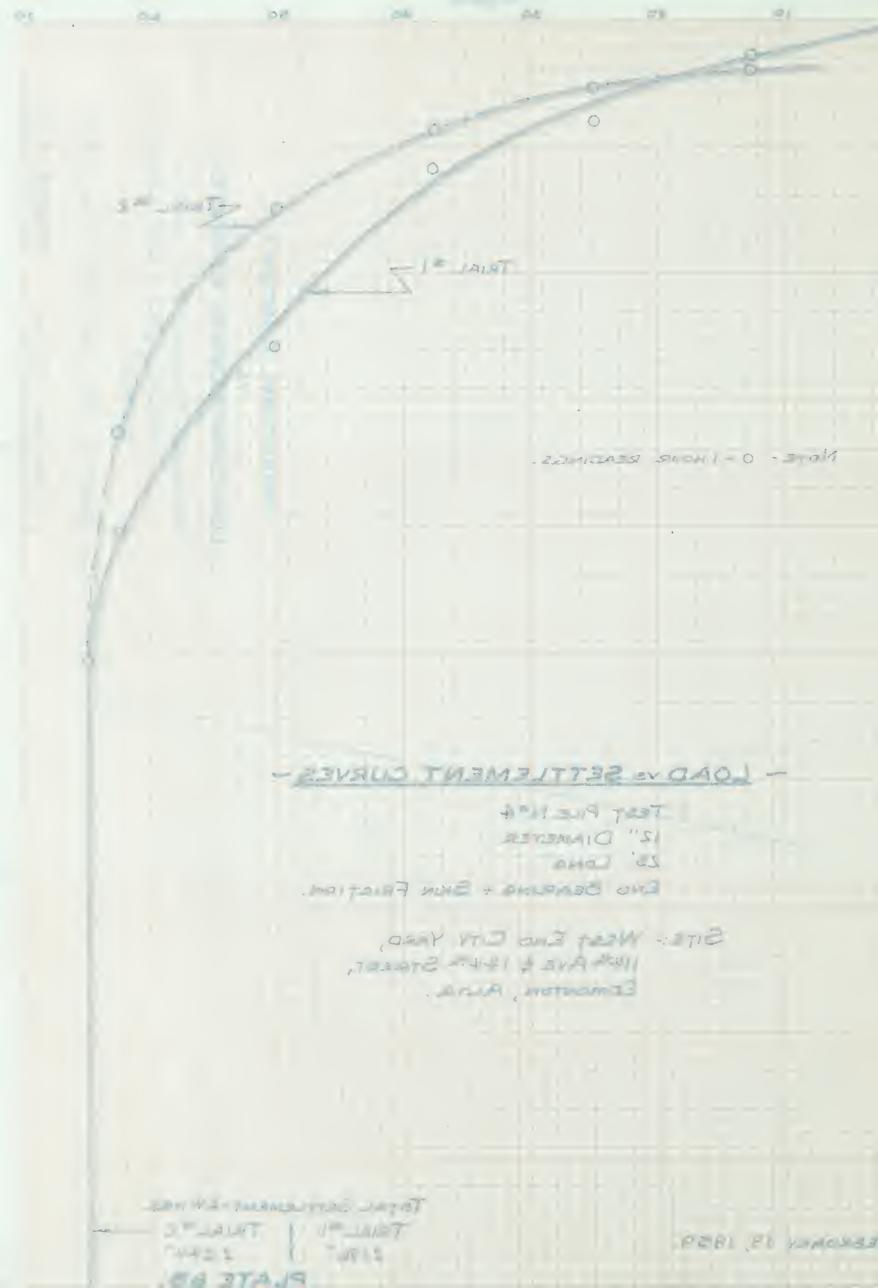


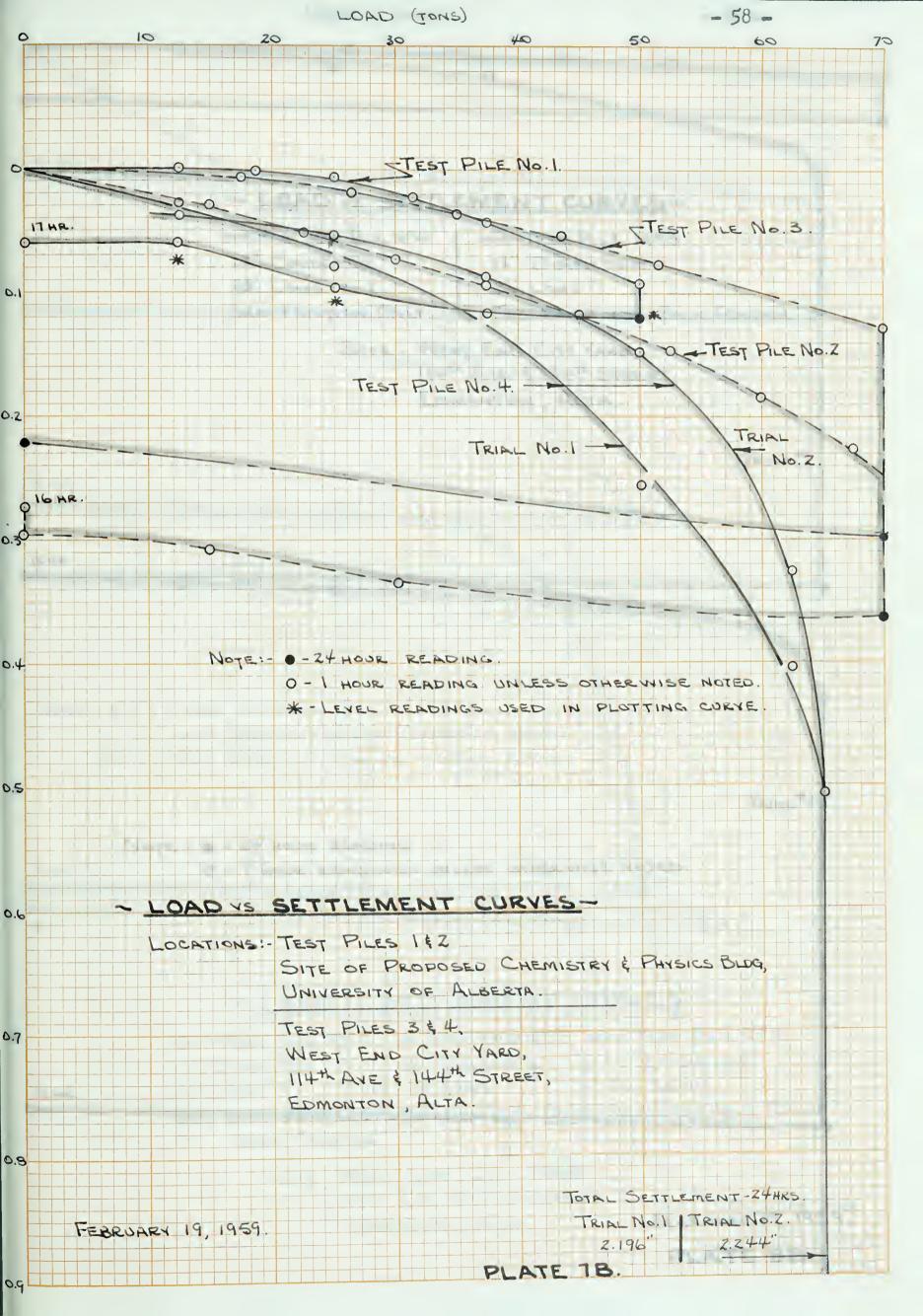
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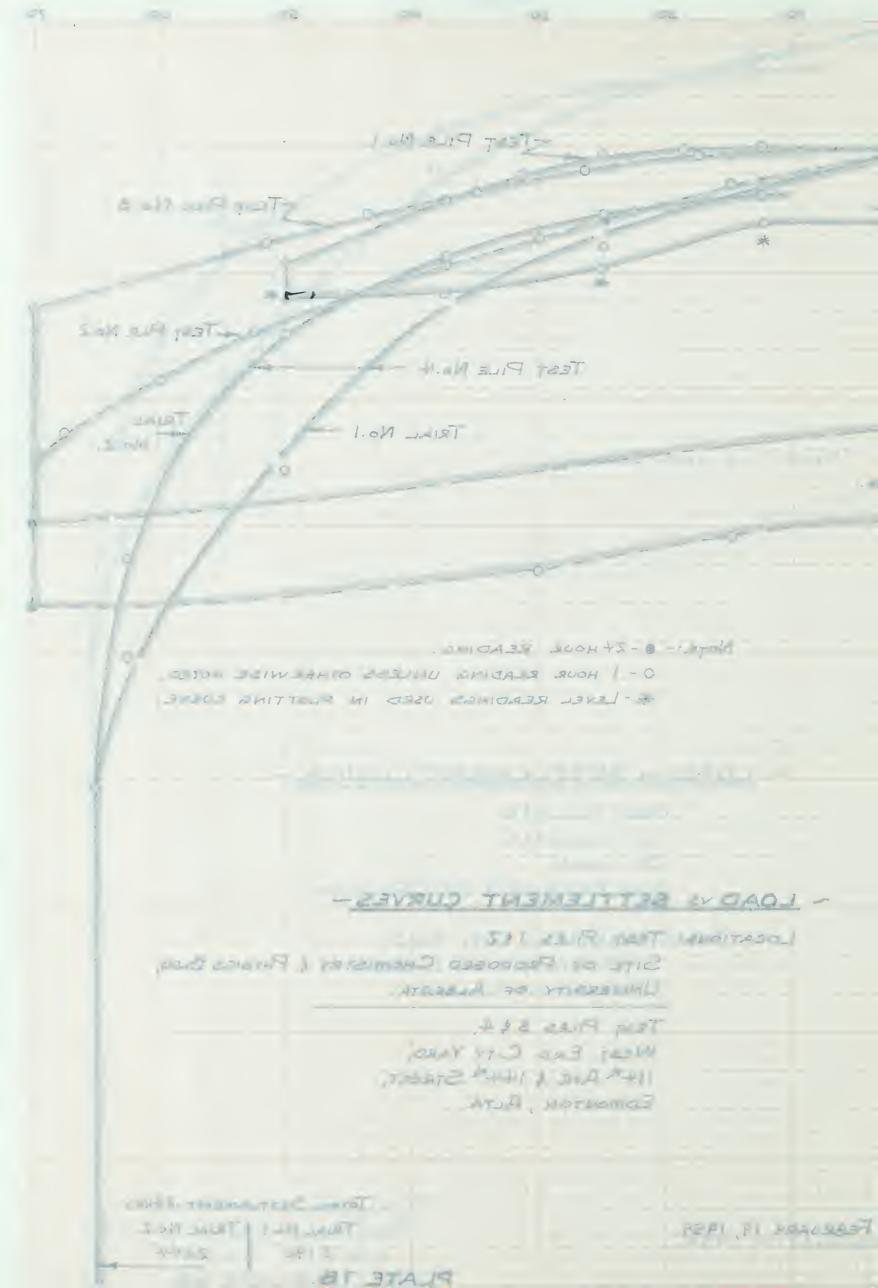


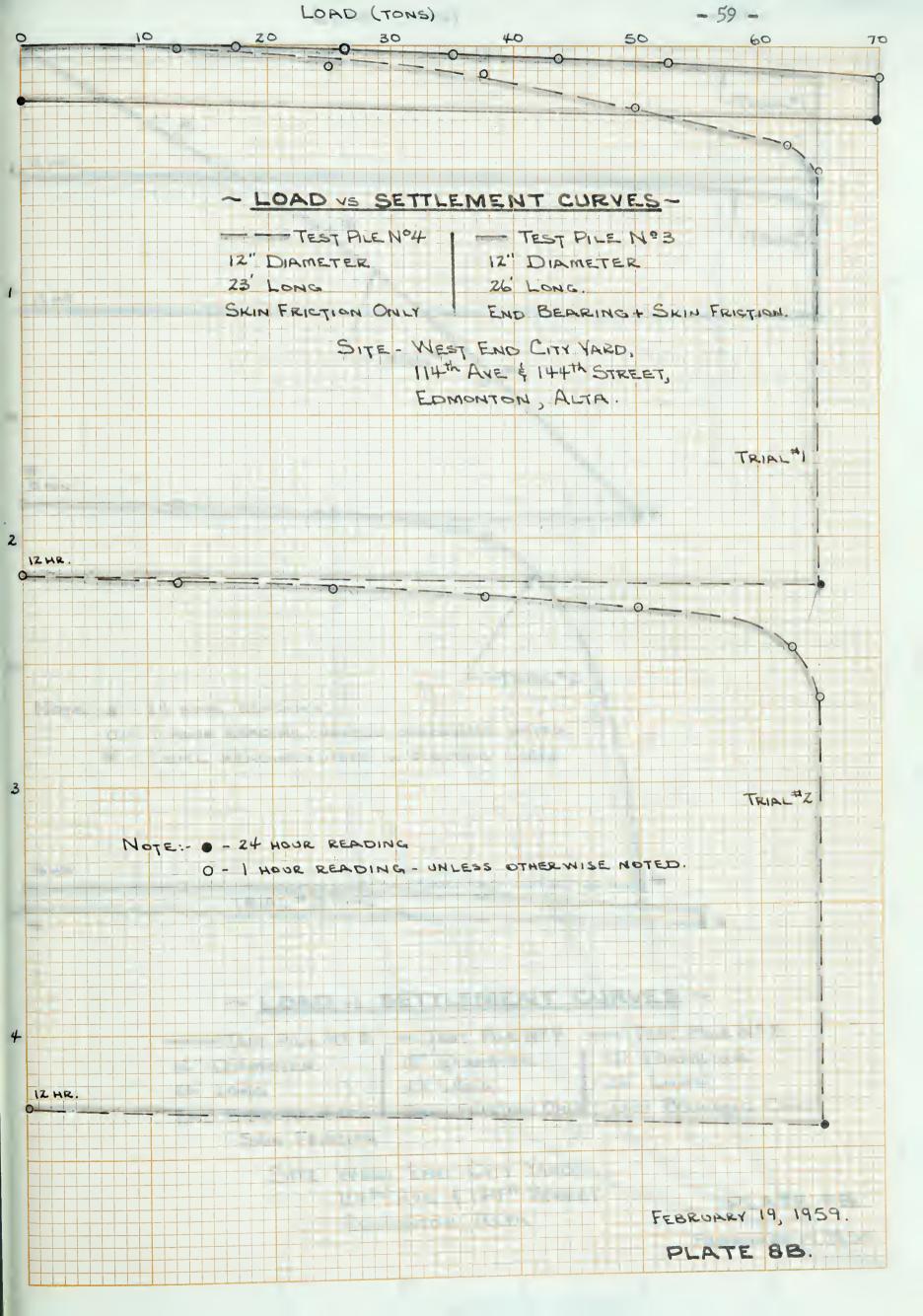
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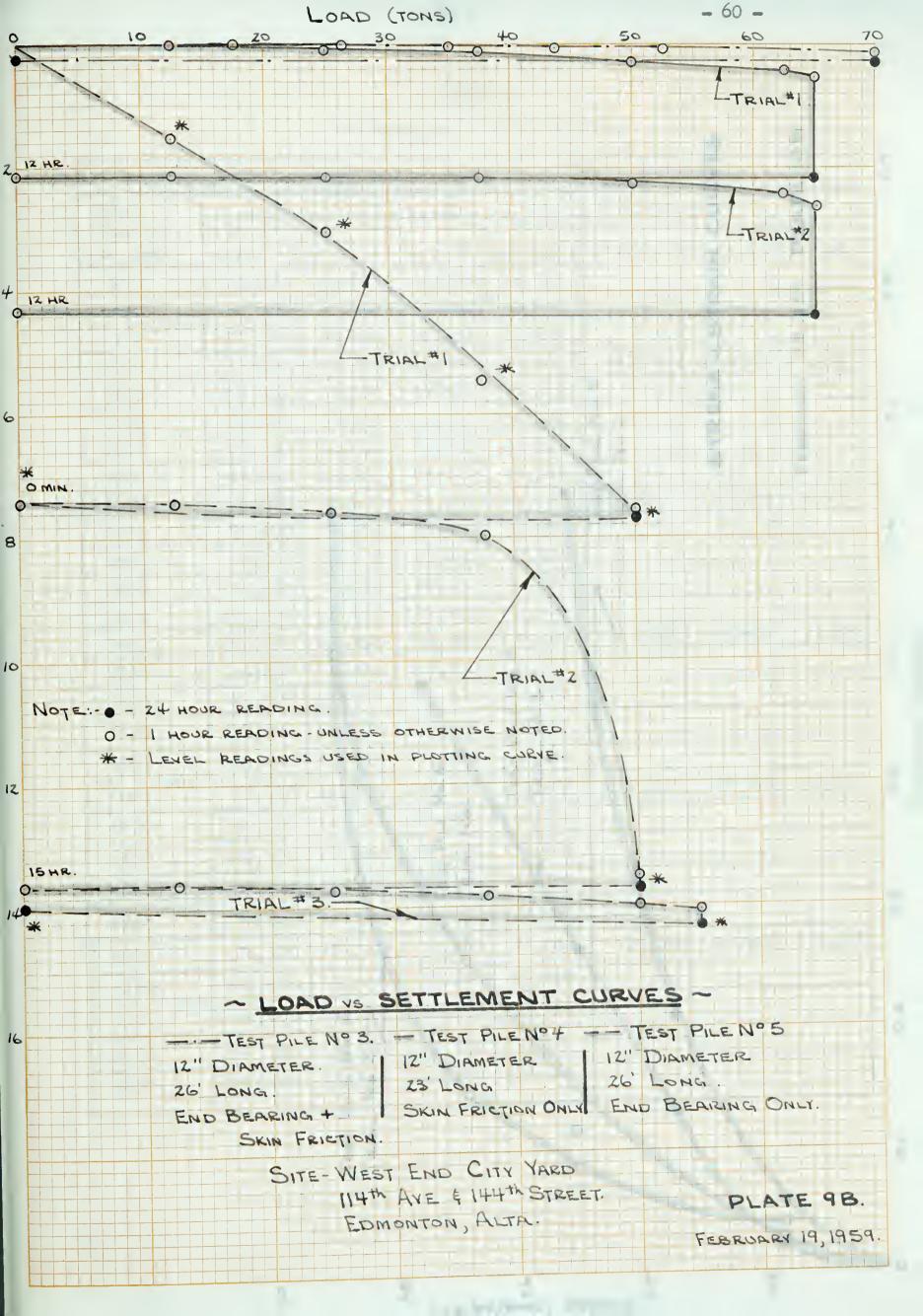
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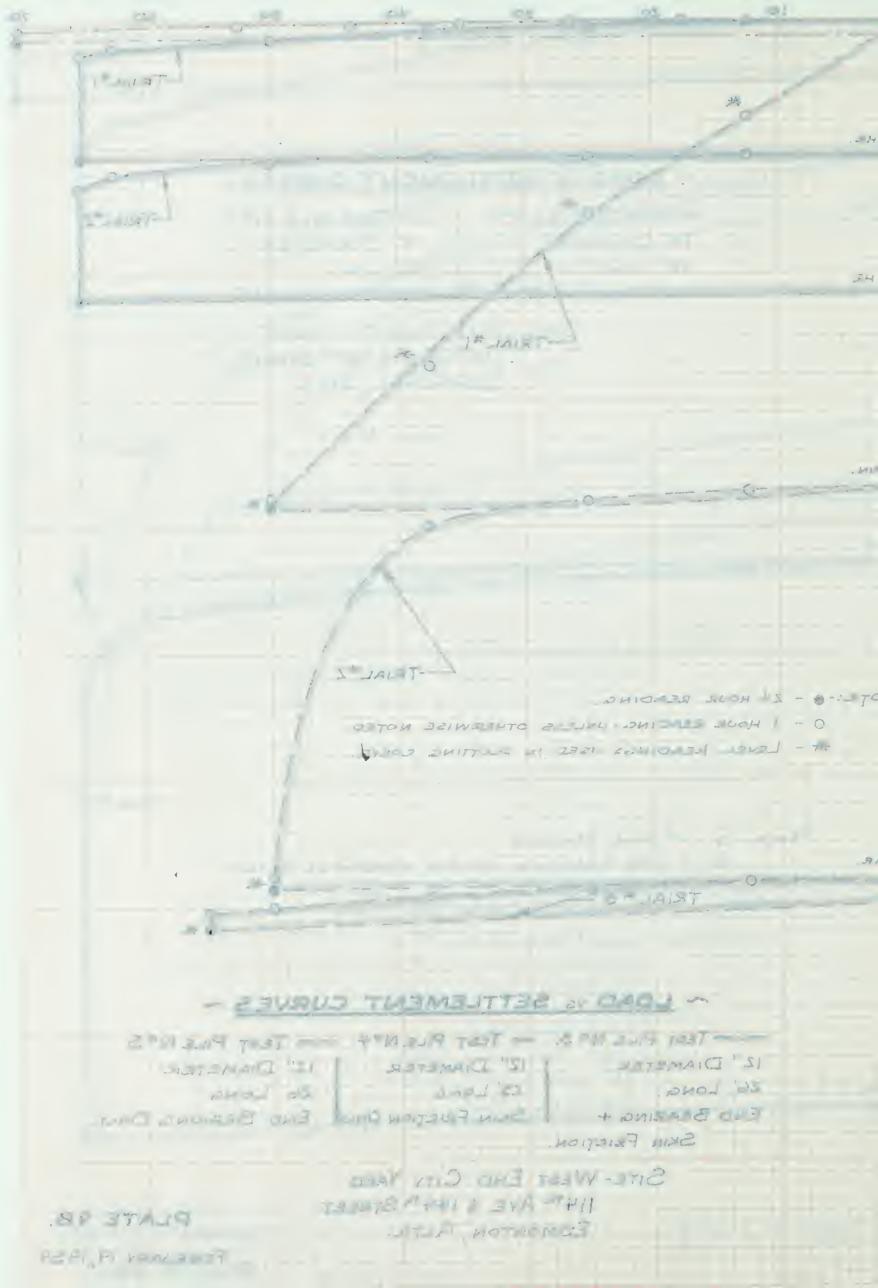
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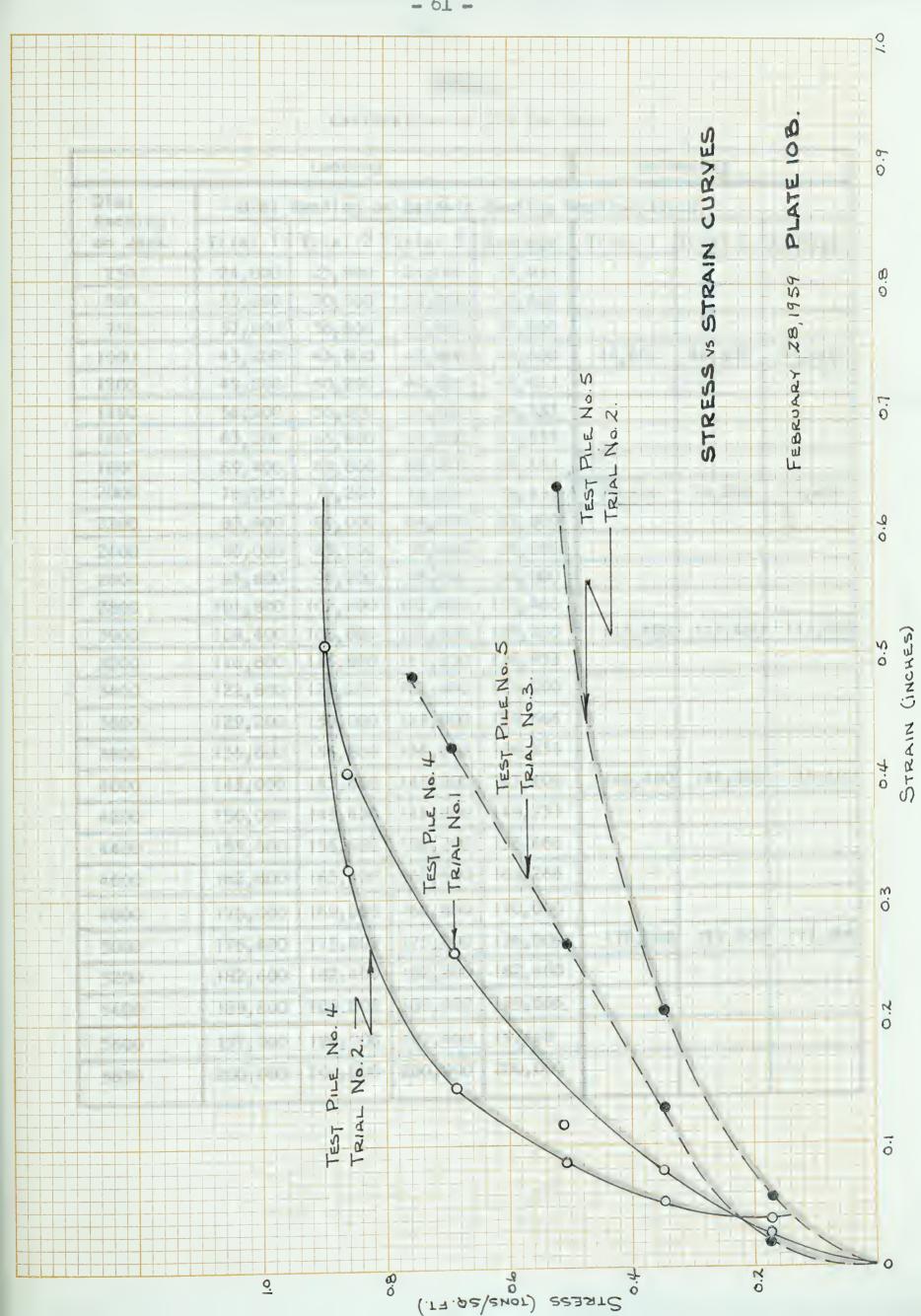
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FEBRUALLY 19 1959.







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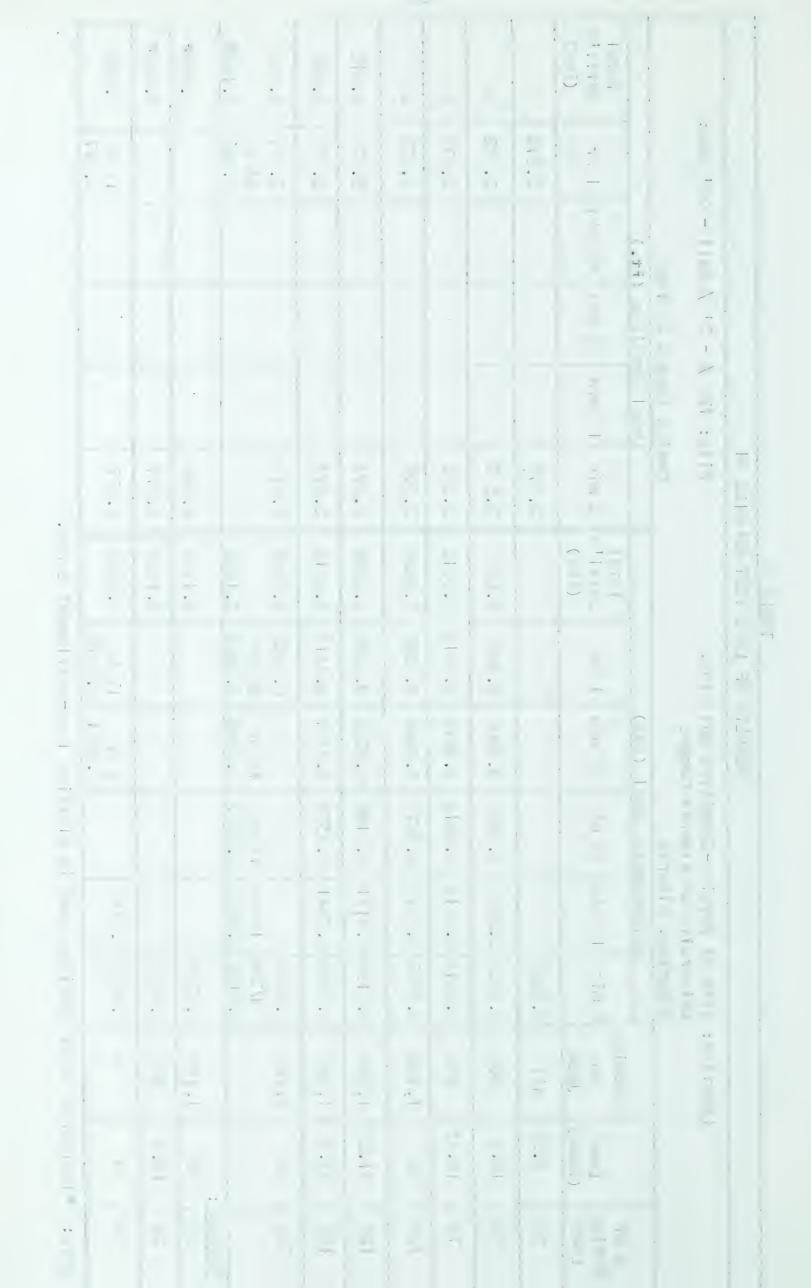
TABLE I
Calibration of 175 Ton Jack

		Loading			Unio	oading					
Dial Reading	Dial	Reading o	n Baldwin	Testing N	Machine (lbs)						
on Jack	Trial I	Trial 2	Trial 3	Average	Trial I	Trial 2	Average				
250	24,000	23,800	24,000	23,933							
500	30,600	30,600	30,800	30,666							
<b>7</b> 50	37,000	36,800	36,600	36,800							
1000	43,200	43,800	43,800	43,600	44,600	44,600	44,600				
1200	49,800	50,200	49,800	49,933							
1400	56,200	56,600	56,800	56,333							
1600	63,200	63,600	63,800	63,533							
1800	69,400	69,800	69,400	69,533							
2000	76,000	76,200	76,200	76,133	79,200	79,200	79,200				
2200	83,400	84,000	84,000	83,800							
2400	90,000	89,200	90,400	89,866							
2600	95,600	96,200	96,200	96,000							
2800	101,800	102,800	102,800	102,466							
3000	109,400	108,800	109,400	109,200	113,400	113,400	113,400				
3200	116,800	116,800	117,200	116,933							
3400	122,800	123,200	123,600	123,200							
3600	129,200	130,000	129,800	129,666							
3800	135,600	136,800	136,800	136,333							
4000	143,000	143,400	143,800	143,400	146,400	146,400	146,400				
4200	150,000	149,400	149,800	149,733							
4400	155,600	156,600	156,200	156,466							
4600	162,800	163,400	163,600	163,266							
4800	170,000	169,800	169,800	170,000							
5000	176,400	175,800	175,800	176,000	178,200	177,600	177,900				
5200	182,600	182,400	182,200	182,400							
5400	189,800	189,800	189,400	189,666							
5600	197,000	198,000	197,400	197,466							
5850	200,000	200,000	200,000	200,000							

E . 1 1 1 , 1 . .

					Total Settl'n (ins)	0	0	0	0	0.012	0.036	0.060	*960°0	*090*0	0.060
		23' deep			_ _	2.572	2.572	2.572	2.572	2.573	2.575	2.577 24 hr 2.582			17 hr 2.577
		Ø bell - 2	ns	(++.)	45 min										
		- 2: 0	= 25 tons	Readings (	30 min										
		: 12" Ø	Design Load	Level Rea	15 min										
	PILE #1	Pile	Desi		O min	2.572	2.572	2.572	2.572	2.573	2.573	2.575	2.580	2.577	2.577
TABLE 2	TEST ON				Total Settlin		00.00	0.0018	0.0085	0.0208	0.0410	0.0902	0.1705	0.1630	0.0578
TAL	OF LOAD	ding,			l hr		0.300	0.3018	0.3085	0.3208	0.3410	0.3902 24 hr 0.4840			3 hr 17 hr 0.0578 0.3641 0.3578
	RESULTS	Physics-Chemistry Building,	Campus	l (ins)	question .		0.300	0.3018	0.3080	0.3202	0.3391	19 hr 0.4748			3 hr 0.3641
		s-Chemis	University of Alberta Campus Edmonton, Alberta	Extensometer Dial	30 min		0.300	0.3018	0.3075	0.3188	0.3372	16 hr 0.4627			
		Physic	Edmonton, Alberta	xtensom	15 min		0.300	0.3018	0.3072	0.3171	0.3341	14 hr 0.4469			
		Site of	Edmonto		0 min - 15 min	0.300	0.300	0.3018	0.3052	0.3135	0.3278	0.3590 4 1/2 hr 0.4182	0.4705	0.4630	0.4320 0.3770
		Location:			Reading on Jack	nil	300	750	1,190	1,560	1,940	2,700	1,190	300	0
		ŭ			Load (tons)	6.25	12.5	18.75	25	31,25	37.5	50	25	12.5	0
					% of Design Load	25	50	75	001	125	150	200	Rebound: 50	25	0

Note: \*Indicates level readings used in plotting load-settlement curve.



				Total Settl'n (ins)		0	0.024	0.048	0.072	960.0	0.120	0.132	0.168	0.348		0.312	0.288	0.252
		23t deep		- hr		3.260	3.262	3.264	3.266	3,268	3.270	3.271	3.274	3.284 24 hr	3.289			3.281
		Bell - 2	(f†)	45 min										3.277   16 1/2 hr	3.288			
		- 21 0	Readings (	30 min							3.270			3.276			5 1/2 h	3.283
		e: 12" Ø	Level Re	15 min														
	PILE #2	<u></u>		0 min	3.260	3.260	3.262	3.264	3.265	3.268	3.269	3.271	3.274			3.286	3.284	3.283
JLE 3	TEST ON			Total Settl'n (ins)		0.0290	0.0506	0.0712	0.0924	0.1178	0.1445	0.1829	0.2276	0.3617		0.3339	0.3084	0.2729
TABLE	OF LOAD	lihg,		-d		0.6196	0.6412	0.6618	0.6830	0.7084	0.7351	0.7735	0.8182	16 1/2 hr 0.9220 -724 hr	0.9523		4	0.0
	RESULTS	ry Buildihg, ampus,	(ins)	4		0.6191	0.6410	0.6608	0.6815	0.7069	0.7350	0.7714	0.8150	0.8900	9381		, c	0.8712
		Physics-Chemistry Bui ty of Alberta Campus, Alberta	ter Dial	i.e		0.6182	0.6391	0.6591	0.6781	0.7041	0.7341	0.7660	0.8110	0.8548				
		Physics-City of Alb	-1 (1)	15 min		0.6170	0.6381	0.6562	0.6750	0.6989		0.7607	0.8004					
		Site of Phy University Edmonton. A		0 min	0.5906	0.6130	0.6324	0.6512	0.6709	0.6930	0.7190	0.7500	0.7907	0.8389		0.9245	0.8990	0.8860
		Location:		Reading on Jack	nil	500	1,050	1,500	1,950	2,400	2,850	3,300	3,750	3,900		1,500	200	0
		07		Load (tons)	7.50	15	22.5	30	37.5	45	52.5	09	67.5	70		Rebound 30	15	0

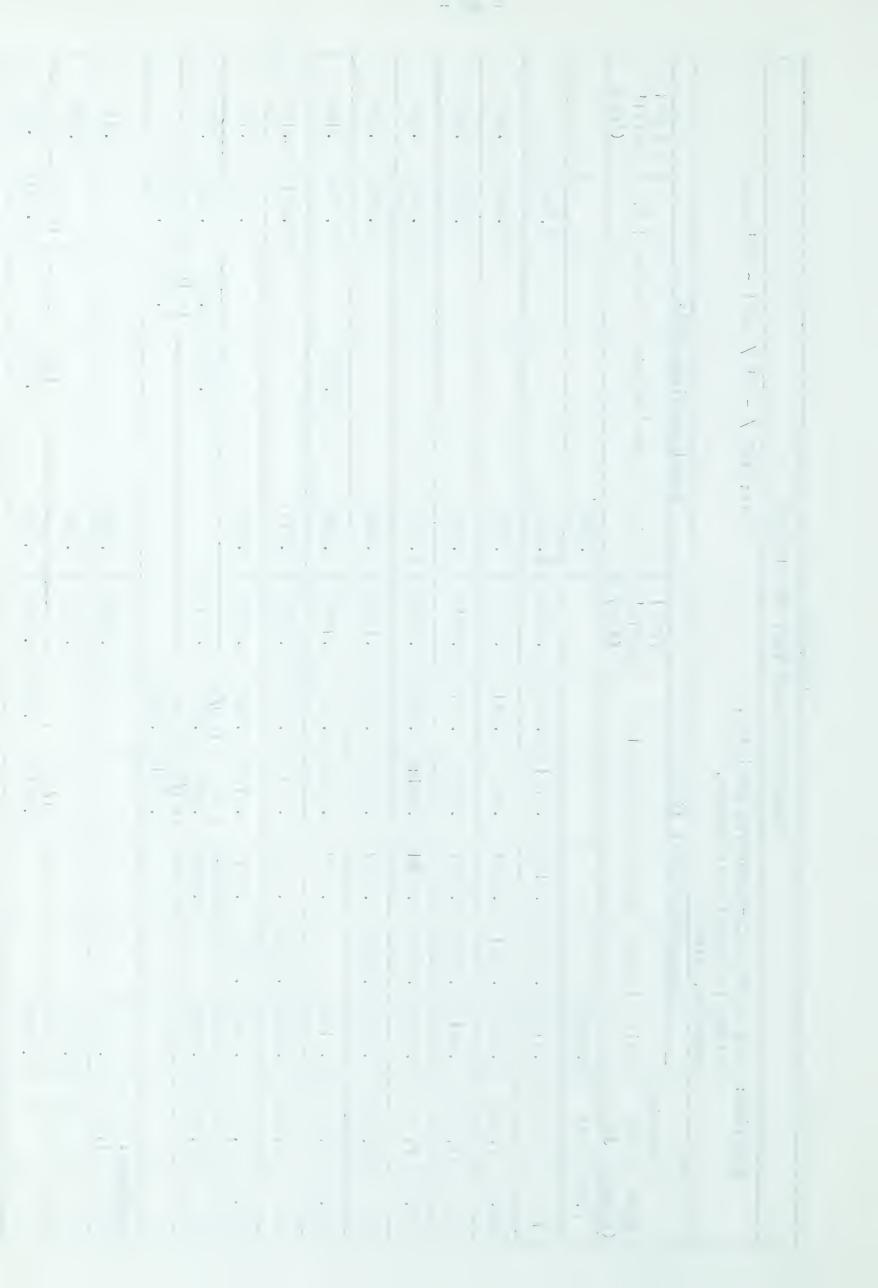
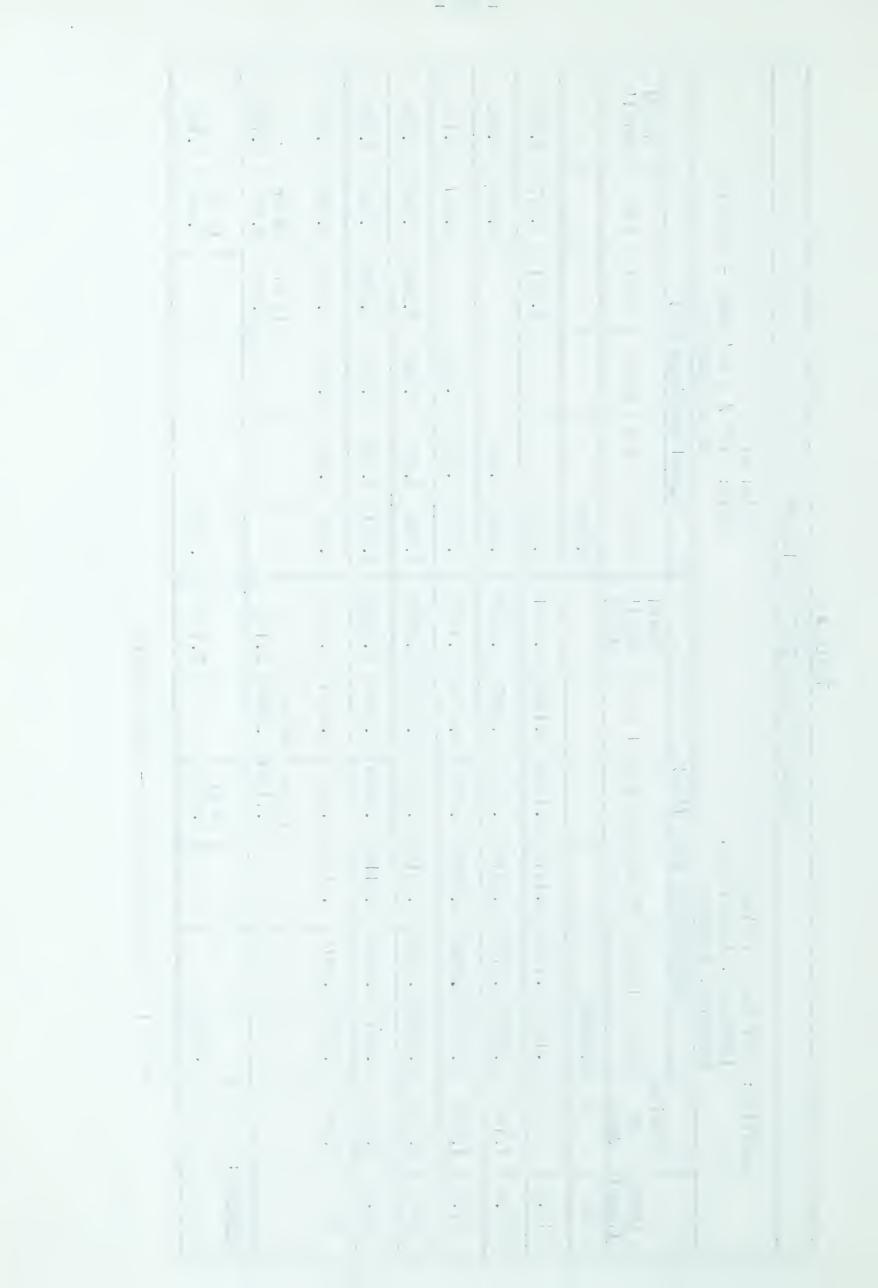


TABLE 4

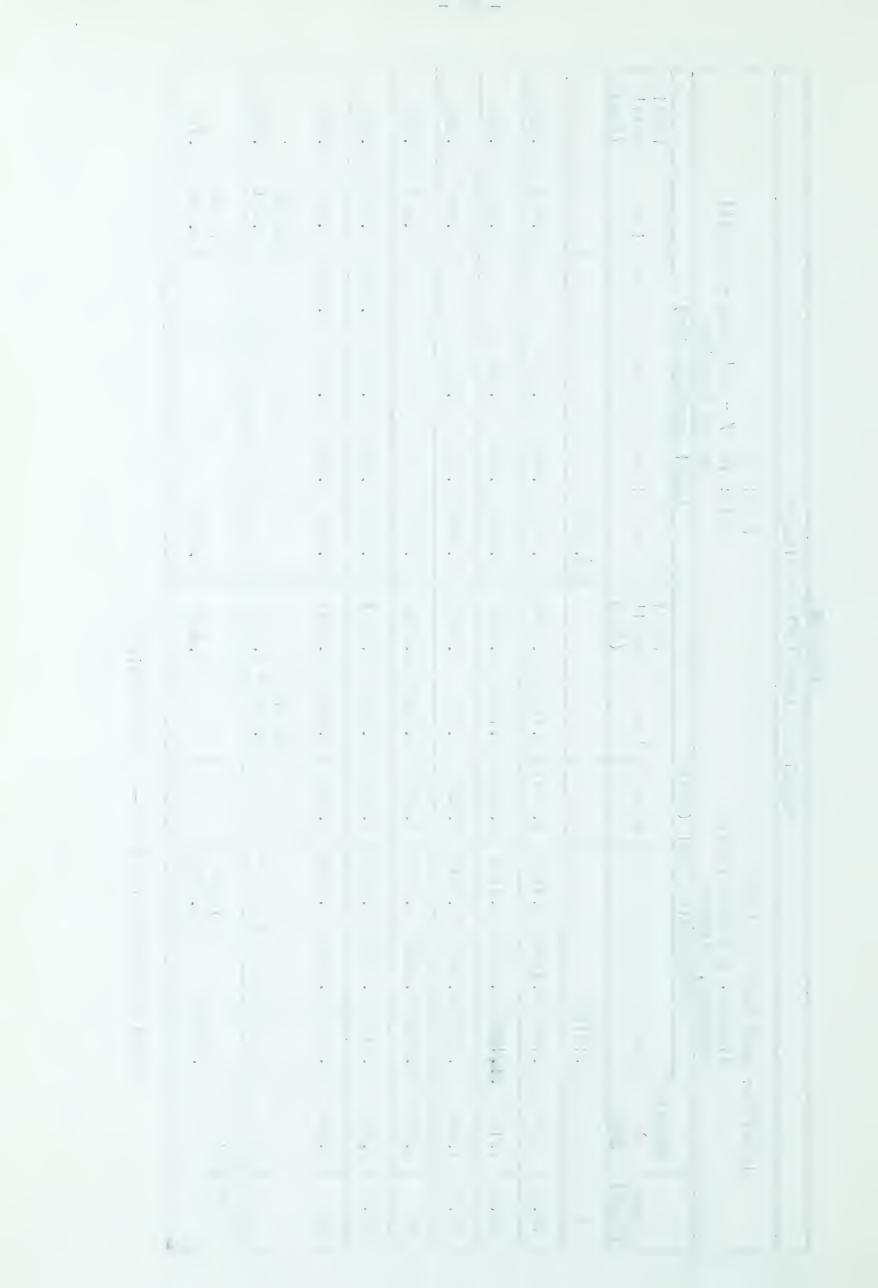
					RESULTS	OF LOAD	TEST	ON PILE #4					
	Location:	-	West End City Yard 114th Ave. & 144th Edmonton, Alberta	Yard 44th St. rta				Trial Pile:	茶	- 231	deep - no bell ing	lo bell	
		ш	Extensometer	eter Dial	(ins)				Level Rea	Readings (	(++)		
Load .	Reading on Jack	O min	15 min	30 min	45 min	l hr	Total Settlin.	O min	15 min	30 min	45 min	- 1	Settlin
0	0	0.1463						3.270					
12.5	300	0.1620	0.1620 0.1690	0.1715	0.1732	0.1733	0.0281	3.270			3.271	3.272	0.024
25.0	1,190	0.2025	0.2150	0.2214	0.2237	0.2262	0.0799	3.274	3.275			3.276	0.072
37.5	1,940	0.2340	0.2340 0.2426	0.2520	0.2578	0.0630	0.1167	3.277	3.278	3.280		3.281	0.132
50	2,700	0.2985	0.3487	0.3918	0.4047	0.4033	0.2570	3.284	3.286	3.288	3.290	3.290	0.240
62.5	3,450	0.4615	0.4873	0.5115	0.5320	0.5497	0.4034	3.291	3.292	3.293	3.296	3.300	0.360
65	3,600	0.5585	0.5815	0.6118	0.6283	0.6527	0.5064	3,305	3,306	3.307	3.308	3.308	0.456
					12 hr 0.1130*	24 hr 0.9500	2.196				12 hr 3.380	24 hr 3.453	2.196
Rebound: 0	0	0.9082			12 hr 0.9001		2.146	3.449				12 hr 3.449	2.148
0	0	0.9082			0.9001		7-140	5.449					0.449

\* Dial ran out during test -- reset reading



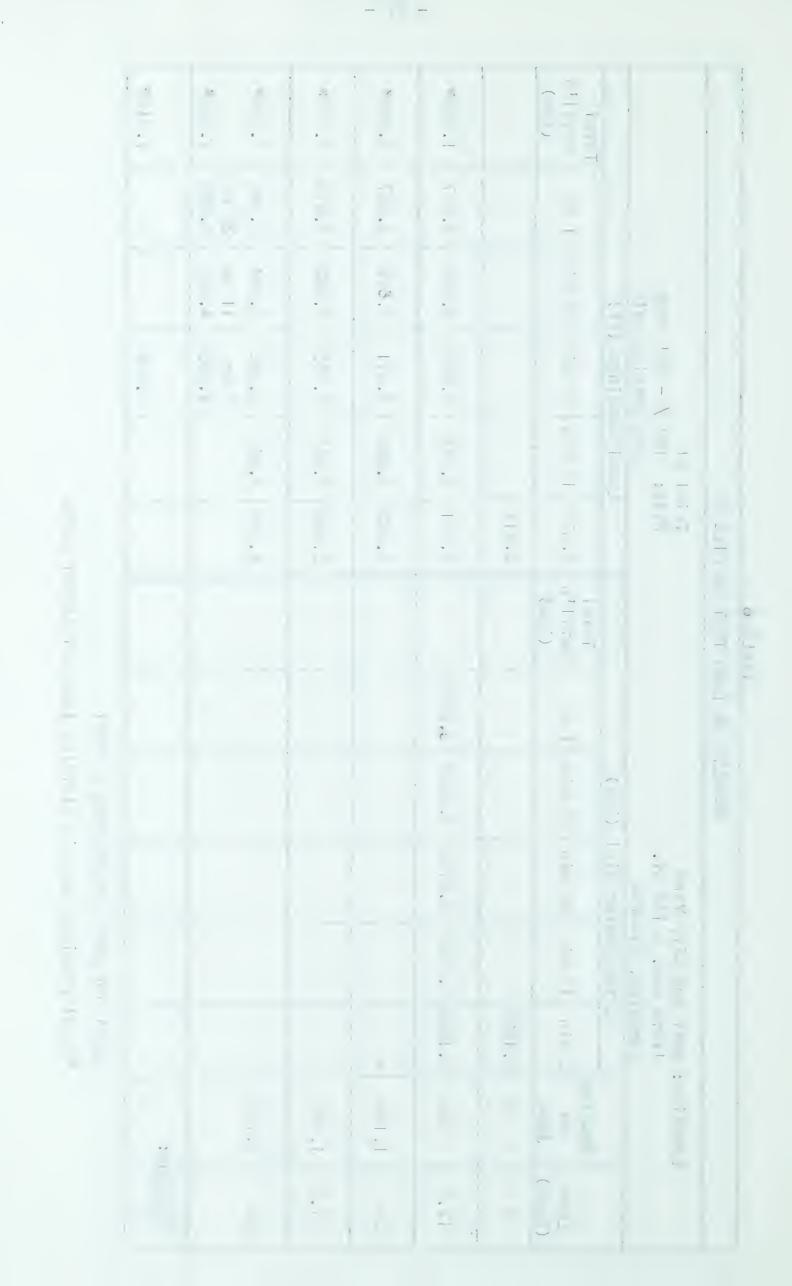
					Total Settl'n (ins)		0.036	0.048	090.0	0.084	0.276	0.480	2.244	2.164
		bell			- pr		3.467	3.468	3,469	3.471	3.487	3.504	24 hr 3.651	12 hr 3.644
		lled on - deeb	6	(++)	45 min						3,485	3.500		
		#2 12" Ø - 23" d	1	Readings	30 min		3.467	3.468	3.469		3,481	3.495		
	4	22	- 1	Level Re	15 min		3,466	3.468	3.468		3.480	3.490		
	PILE #4	Trial Pile:			O min	3,464	3.466	3.467	3.468	3.470	3.472	3.487		3.645
TABLE 5	D TEST ON				Total Settl'n (ins)		0.0394	0.0522	0.0856	0.1486	0.3251	0.5077	2.244	2.164
<b> -</b>	OF LOAD				- r		0.1812	0.1940	0.2274	0.2904	0.4669	0.6495	24 hr 0.4720*	
	RESULTS		- 1	(108)	45 min		0.1811	0.1940	0.2245	0.2841	0.4343	0.6025		
		City Yard & 144th Street		eter Diai	30 min		0.1810	0.1931	0.2215	0.2741	0.3940	0.5595		12 hr 0.3915
		West End City Yard 114 Ave. & 144th S	Edmonton, Alberta	Extensometer	15 min		0.1796	0.1915	0.2165	0.2624	0.3622	0.5180		
			Edmont	11	0 min	0.1418	0.1795	0.1902	0.2095	0.2460	0.3115 0.3622	0.4767		0.3925
		Location:			Reading on Jack	0	300	1,190	1,940	2,700	3,450	3,600		o • pur
					Load (tons)	0	12.5	25.0	37.5	50	62.5	65		Rebound 0

\*Dial ran out during test -- reset reading



				Total . Settlin (ins)		1.560*	3.040*	5.436*	7.536*	7.416*
				- hr		3.547	3,687	3.870	4.045 24 hr 4.055	
		deep	(ft)	45 min		3.546	3.678	3.862	4.044    hr 4.055	
				30 min		3.545	3.671	3.855	4.042 3 hr 4.053	4.035
	2	MC.	Level Readings	15 min		3.544	3,660	3.845	4.040	
	ON PILE #5	Trial Pile:		O.min	3.417	3.53	3.642	3.820	4.036	
TABLE 6				Total Settiin (ins)						
TA	RESULTS OF LOAD TEST			l hr		0.2888				
	RESULTS		(ins)	45 min		0.2839				
		Yard 144 St. erta	her Dial	I .		0.2728				
		West End City Yard 114th Ave. & 144 St. Edmonton, Alberta	Extensometer	15 min		0.1290 0.2549				
		ſ	EX	O min	0.1923	0.1290	+			
		Location:		Reading on Jack	0	300	061,1	1,940	2,700	0
				Load (tons)	0	12.5	25	37.5	50	Rebound

+Dial not used for balance of test
\*Level readings used in plotting load settlement curve



Г											
				Total Settlin (ins)		090.0	0.180	0.588	6.110*		<b>*</b> 081 <b>*</b> 9
				l hr			4.055	4.089	4.550	بر ح. ر	
		eep n l y	+)	45 min			4.055	4.086			
		Trial #2 Pile - 12" Ø - 26' deep End bearing only	Readings (ft)	30 min			4.055	4.084	.24 hr 4.570		
	¥5	- 12" Ø End be	Level Rea	15 min			4.053	4.080	17 1/2hr24 hr 4.565   4.570		
	ON PILE	Trial #2 Pile - I	) Te	O min	4.040	4.045	4.050	4.072	4.532		4.559
TABLE 7				Total Settl'n (ins)		0.0548	0.2067	0.6357			
	RESULTS OF LOAD TEST			l hr		0.2903	0.4422	0.8712	0.7040+		
	RESUL		(ins)	45 min		0.2903	0.4379	0.8200		ر د د	0.8154
		Yard 144th St. erta	10	30 min		0.2903	0.4302	0.8022	-24 hr 0.9892		
		West End City Yard 114th Ave. & 144th Edmonton, Alberta	Extensometer Dial	15 min		0.2760 0.2880	0.3860 0.4158	0.7507	17 1/2 hr 24 hr 0.8927   0.989		
			Ext	O min	0.2355	0.2760	0.3860	0.6645			0.8382
		Location:		Reading on Jack	0	300	1,190	1,940	2,700	p	0
		_		Load (tons)	0	12.5	25	37.5	50	Rebound	0

+Dial ran out during test -- reset reading

\*Level readings used in plotting load settlement curve

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					RESULTS	OF LO	ABLE ON TEST ON	PILE #5	10				
	Locations		West End City Yard 114th Ave. & 144th Edmonton, Alberta	Yard 144th St. erta				Trial Pile:	类	%3  2" Ø - 26' deep End bearing only	deep only		
		Ê	xtensome	Extensometer Dial	(ins)				Level Readings		(++)		
Load (tons)	Reading on Jack		0 min - 15 min	30 min	45 min	l hr	Total Settlin (ins)	O min	15 min	30 min	45 min	l hr	Total Settiin (ins)
0	0	0.7272						4.590					
12.5	300	0.7135	0.7135 0.7098	0.7088	0.7084	0.7072	0.0200	4.592	4.592	4.592	4.592	4.592	0.024
25	1,190	0.6072	0.5988	0.5942	0.5926	0.5892	0.1380	4.600	4.600	4.600	4.601	4.601	0.132
37.5	1,940	0.4988	0.4988 0.4759	0.4694	0.4656	0.4642	0.2630	4.608	4.609	4.609	4.609	4.609	0.228
50	2,700	0.4027	0.3561	0.3332	0.3150	0.3037	0.4235	4.616	4.619	4.620	4.622	4.623	0.396
55	3,000	0.2610				0.4720+		4.627				4.630 24 hr 4.651	0.480
Rebound: 0	0 : pu	0.8335				24 hr 0.8690		4.623				24 hr 4.622	0.384*

+Dial ran out during test -- reset reading

\*Level readings used in plotting load settlement curves

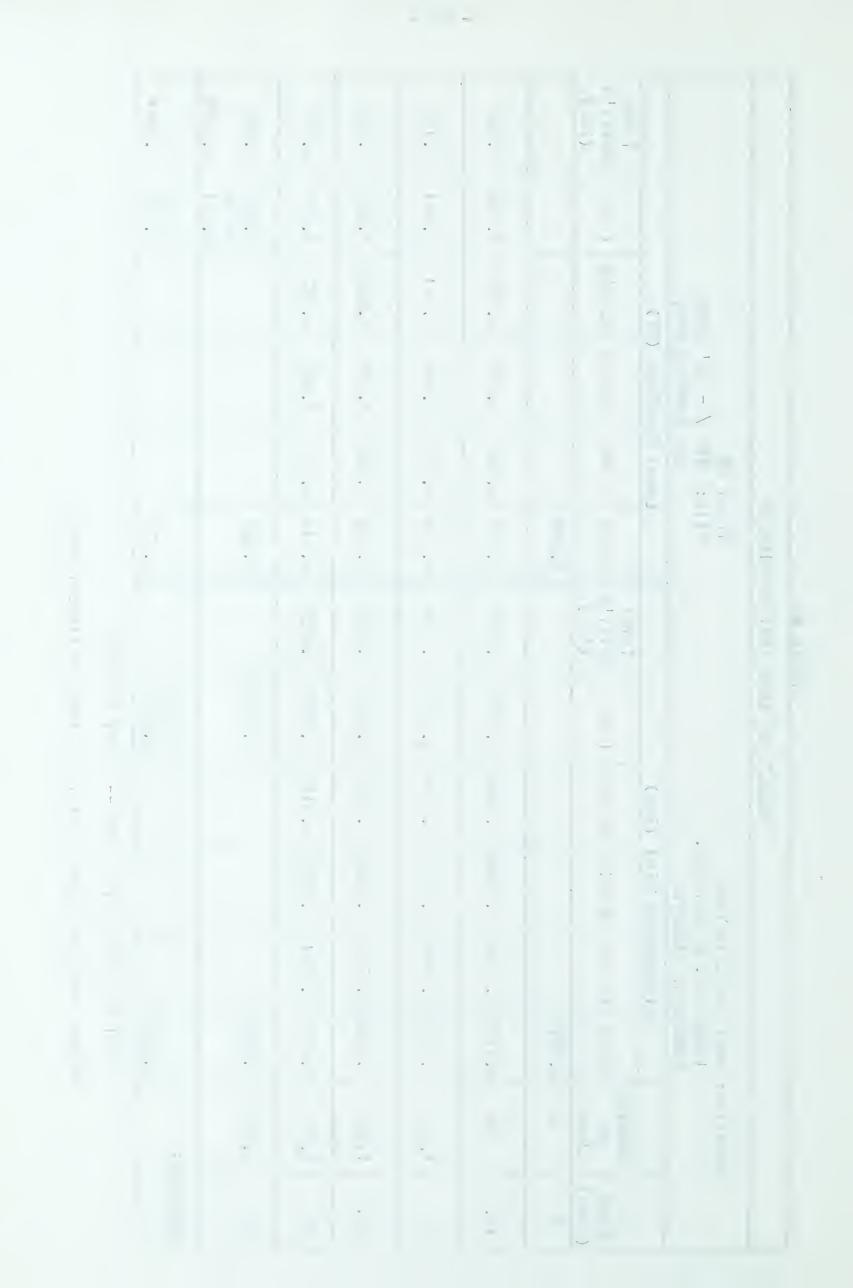
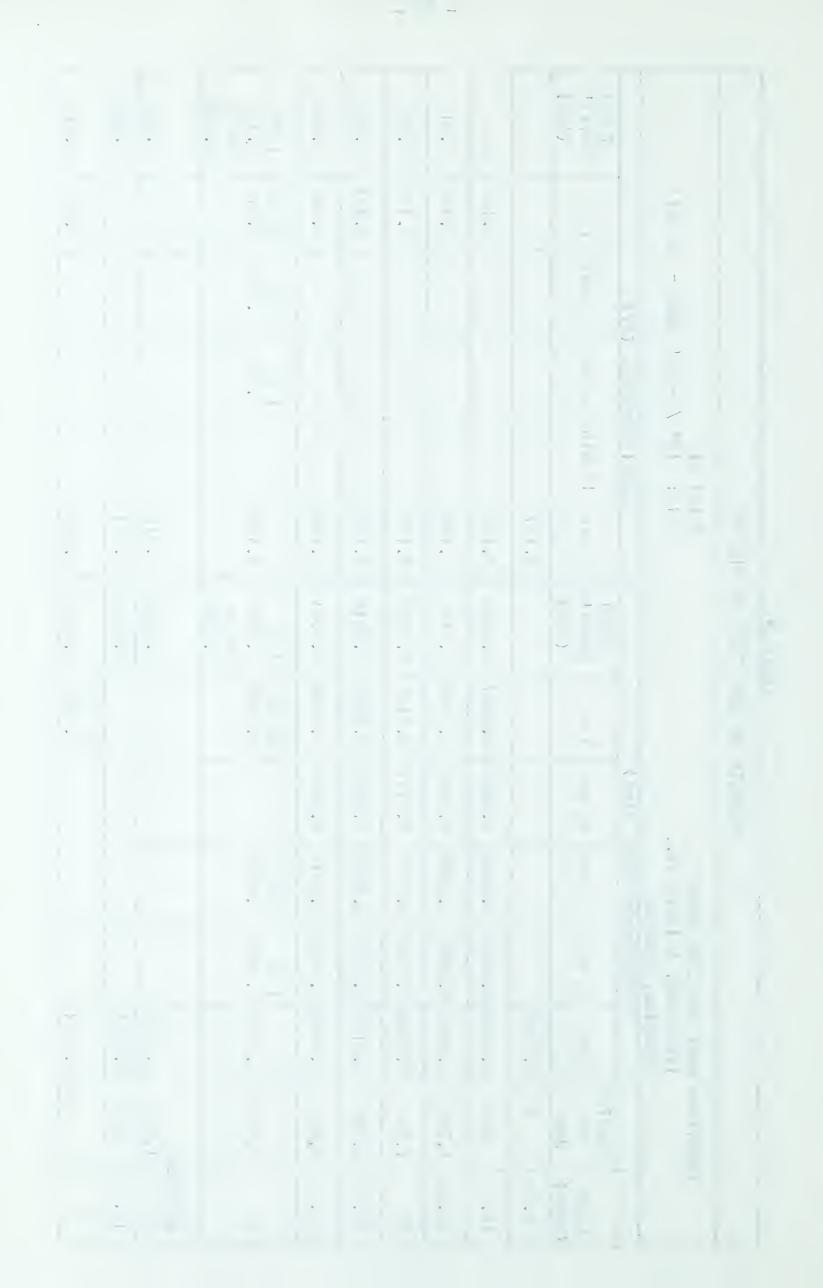


TABLE 9

			- Lu ( ( ( )			21							
			Sett!		0	0.012	0.036	0.048	0.072	0.156 24 hr	0.300	0.288	0.216
	lo bell		l hr		3.187	3.188	3.190	3.191	3.193	24 hr 3.214			24 hr 3.205
	ı dəəp	(++)	45 min							5 hr 3.203			
	Ø - 26' deep - no bell	Readings (	30 min							1 hr 3.200			
	#   2"	Level Rea	15 min										
PILE #3	Trial		0 min	3.187	3.187	3.188	3.189	3.190	3.192	3, 196	3.212	3.211	3.210
AD TEST ON			Total Settlin		0.0070	0.0198	0.0375	0.0514	0.0771	1 hr 0.1298 24 hr 0.2995	0.2888	0.2764	0.2221
OF LOAD			- hr		0.2837	0.2965	0.3142	0.3281	0.3538	24 hr 0.5762			24 hr 0.4988
RESULTS		(ins)	45 min		0.2829	0.2955	0.3119	0.3270	0.3529				
	Yard 44th St., erta	eter Dial	30 min		0.2805	0.2938	0,3103	0.3240	0.3512	5 hr 0.4530			
	West End City Yard 114th Ave. & 144th Edmonton, Alberta	Extensometer	IS min		0.2805	0.2920	0.3069	0.3229	0.3437	i hr 0.4065			
	West En 114th A Edmonto	T)	O min	0.2767	0.2769	0.2892	0.3020	0.3192	0.3368	0.3775	0.5655	0.5531	0.5371
	Location:		Reading on Jack	0	650	1,250	1,800	2,300	2,850	3,900	1,800	650	0
	Lc		Load (tons)	8.75	17.5	26.25	35	43.75	52.5	70	Rebound 35	17.5	0



### APPENDIX A

# LABORATORY TEST RESULTS

SITE OF CHEMISTRY AND PHYSICS BUILDING



UNIVERSITY	of ALE	BERTA		T TEST PI			
				CHEM-PI	HYSICS	Built	ING.
DEP'T of CIVI			SAMPLE				
SOIL MECHANIC				#Z			
ATTERBER	G LI	MITS	TECHNI	CIAN P.		DEPTH DATE	2
	lio	uid Limit	TILOTHI	OTAN 1.7		DAIL	
Trial No.	1	2	3	,	2		3
No. of Blows	41	41	38	14	13		16
Container No.	V79	V46	V20	V71	V44		V36
Wt. Sample Wet + Tare	93.7315	99.5072	82.0329	86.0770	81.76	00 8	2.3058
Wt Sample Dry + Tare	89.7084	95.0826	17.5017	81.9094	78.10		7.7132
Wt. Water	4.0231	4.4246	4.5252	4.1676	3.6		4.5926
Tare Container	79.5047	83.7237	65.5906	12.0433			7.0527
Wt of Dry Soil	10.2037	11.3589	11.9371	9.8661			0.6605
Moisture Content w%	The second secon	39.0	38.0	42.3	43.		4-3./
				Plastic	Limit		
	<del>·├──├──┞─╃╼╂┈</del> ┼┫	age Values	Trial No.	1140110	/	Z	3
	w	1= 40.6 %	Container	No.	4	7	11
		D= 19.4%	Wt. Sample		32.4303	32.804	5 42.3165
		•	Wt. Sample				
	W <sub>s</sub>		Wt. Water				4 0.5182
	Ip	= 21.2 %	Tare Cont	giner			4 39.1062
			Wt. of Dry				7 2.6921
			Moisture C	ontent %	19.Z	19.6	1
	111111111111111111111111111111111111111	= 2.26	R	Shrinkage	Limit		
44			Trial No.				
			Container	No.			
			Wt Sample				
43			Wt Sample				
			Wt-Water				
842			Tare Cont	ainer			
			Wt. of Dry	Soil Wo			
É			Moisture Co	ontent w%			
÷41			Vol. Contai				
Content			Vol. Dry So				
			Shrinkage				
40			Shrinkage	Limit Ws			
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539				= 40- (-	Wo		
Mo							
<u> </u>			Descripti	on of Sam	iple:		1 1 1
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37			#				
			Remarks:				
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UNIVERSITY of ALBERTA
DEP'T of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
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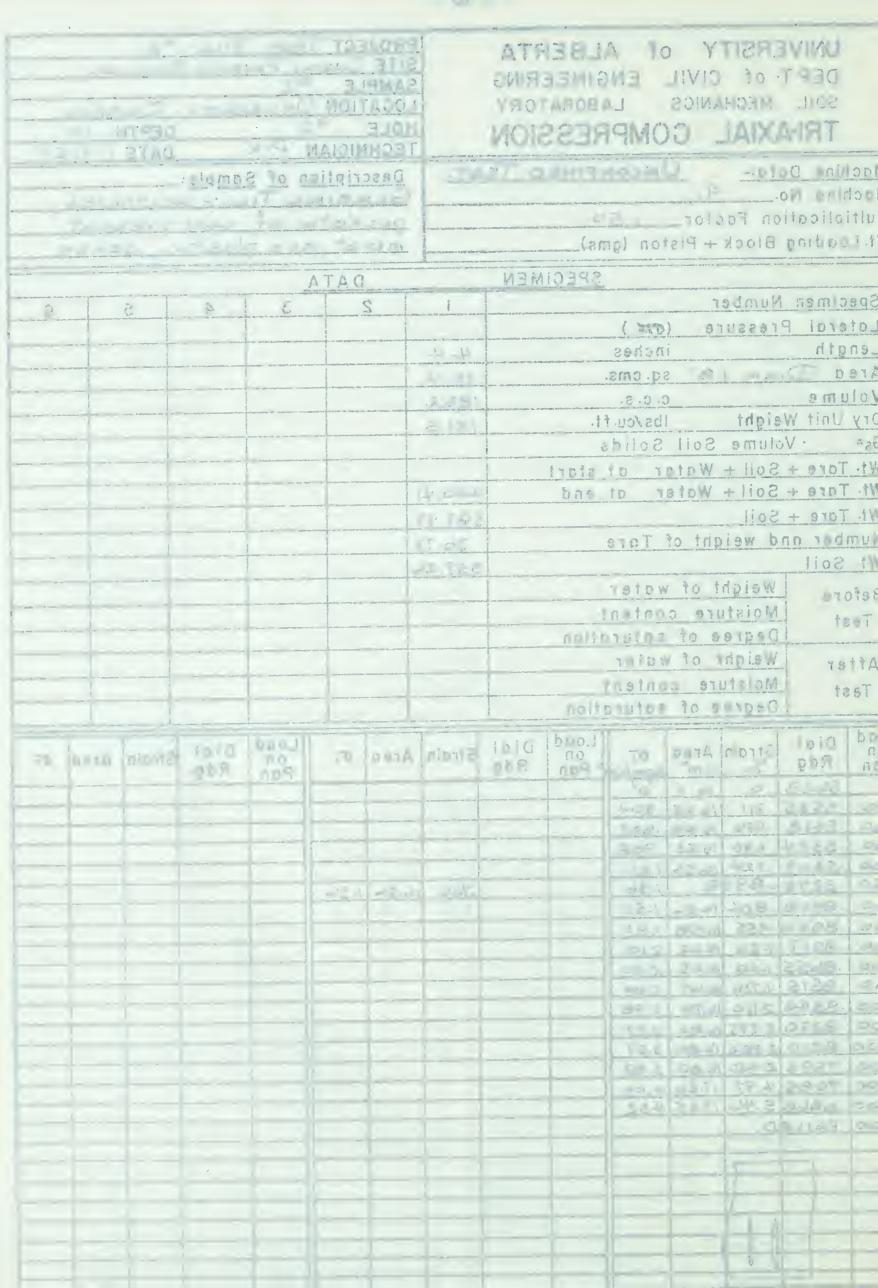
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Load on Pan	Dial Rdg .0216 .0169	Moistu Degre Strain 70 0 .150	Area cm <sup>2</sup> 9.93 9.94	ontent satura o, Kam/cm	Load		25.z	%		6.	on		Strain	Arec	57
Load on Pan	Dial Rdg .0216 .0169 .0109	Strain To O .150 .340	Area cm 9.93 9.94 9.96 9.98	ontent satura or Kgm/cm o a.zol o.4az	Load		25.z	%		57	on		Strain	Arec	57
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Tes Load on Pan  0 9ms 20 40 60 80 100 120 140	Dial Rdg .0216 .0169 .0109 .0059 .9965 .9945	Moistu Degre Strain %0 0 .150 .340 .500 6.800 5.803 6.803	9.93 9.94 9.96 9.98 10.01 10.02	ontent satura or Kym/cm o a.zol a.yoz a.6oz a.8oo 1.0oo 1.193 1.393	Load		25.z	%		51	on		Strain	Arec	57
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1336084 UNIVERSITY OF ALBERTA SATE SAMPLE LOGATION DERT OF GIVIL ENGINEERING SDIL MECHANICS LABORATORY TRIAMAL COMPRESSION Orieription of Semeler achine Data:ochine No. ultiplication Factor Charles Tanasa t Leding Block + Piston (gms). SPECILER DLITA E 1 0 redmud n. miosac \* 1 \_dieral Pressure (ors) .ength inenes 31.0 no or a more to a glass red en el el sq. cms. emuloi olume cc.s. Ory Unit Weight lbs/cu.ft. C C.S. 2 791 se l'olume Soil Soilds Vi. Tare + Soil - Water of start VI. Tura + Soll - Water J end ATALL 1102 - 930T AV lumb rand licht of tale Mac .tv Jr 30 Weight of wo erore factnos erut ioM 1257 Dear of saturation Weight of weld: 20.18 After Misture conten Test D gras of soturotion bo Loud Dial Storin Area P.0.070 Inla Strain Area U to pass night? 000 Rog 0.07 POO DOR and the second s Alto. MALO STEE CONTRACTOR ---Was sign os in Fada OUT OF HIGH COMPANDED . THE SEA LONG THE P. 143 1535 1553 MILE 0 BIEN PEOPLE (OLBINE j.Cthe state of the s per les oil savil or ep. WELL GOOD SHULLINGS TRULADIA MILITARE -03 7.100 M 281 1 3681 DB 1103

UNIVERSITY of ALBERTA DEPT of CIVIL ENGINEERING SOIL MECHANICS LABORATORY TRI-AXIAL COMPRESSION	PROJECT TEST PILE # Z SITE CHEM- PHYSICS BUILDING. SAMPLE # Z LOCATION UNIVERSITY CAMPUS. HOLE # Z DEPTH 10' TECHNICIAN P.K. DATE 1/9/59							
Machine Data: UNCONFINED TE	ST.							
Machine No9		ST. Description of Sample: GLACIAL TILL: weathered						
Multiplication Factor ×50		pockets of coal present,						
Wt. Loading Block + Piston (gms.)		moi	st, n	on p	asti	c, dens	se.	
SPECIMEN	DATA							
Specimen Number	-	2		3	4	5	6	
Lateral Pressure (67%)								
Length inches	4.4							
Area Diam 1.8" sq. cms.	16.4							
Volume c.c.s.  Dry Unit Weight lbs/cu-ft-	183.2							
Gs= · Volume Soil Solids	121.5							
Wt. Tare + Soil + Water at start		1						
Wt. Tare + Soil + Water at end	440.4	-1						
Wt. Tare + Soil	387.9	9						
Number and weight of Tare	30.7							
Wt. Soil	357.2	.6						
Before Weight of water								
Test   Moisture content   Degree of saturation								
Wainha of water								
Moisture content					<del></del>			
Test Degree of saturation								
_oad Dial     Load Dial				Load	Dial			
Load Dial Strain Area G Load Dial	1.3 11 (121)	Area	G.	Load on Pan	Dial Rdg.	Strain Ar	ea or	
ond Dial Strain Area of Load Dial on Rdg % Cm² kgm/cm² Pan Rdg	1.3 11 (121)	Area	G.	on		Strain Ar	ea 67	
On Rdg Strain Area of Load Dial on Rdg % Cm² kam/cm Pan Rdg  O .5628 O 16.4 O	1.3 11 (121)	Area	67,	on		Strain Ar	ea or	
Ond Dial Strain Area of Load Dial Pan Rdg % Cm² kgm/cm² Pan Rdg O .5628 O 16.4 O 100 .5535 .211 16.45 .304 200 .5418 .476 16.48 .607	1.3 11 (121)	Area	67.	on		Strain Ar	ea or	
Load Dial Strain Area of Cm² kgm/cm² Pan Rdg  O .5628 O 16.4 O  100 .5535 .211 16.45 .304  200 .5418 .476 16.48 .607  300 .5324 .690 16.52 .908  400 .5309 .724 16.55 1.21	Sirdir			on		Strain Ar	ea or	
Load Dial Strain Area of Rdg  On Rdg % Cm² kgm/cm Pan Rdg  O .5628 O 16.4 O  100 .5535 .211 16.45 .304  200 .5418 .476 16.48 .607  300 .5324 .690 16.52 .908  400 .5309 .124 16.55 1.21  450 .5295 = 8938 1.36	Sirdir	Area		on		Strain Ar	ea or	
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Load Dial Strain Area or kgm/cm Pan Rdg  O .5628 O 16.4 O  100 .5535 .211 16.45 .304  200 .5418 .476 16.48 .607  300 .5324 .690 16.52 .908  400 .5309 .724 16.55 1.21  450 .5295 = 8938 1.36  500 .8908 .824 16.56 1.51  500 .8850 .955 16.58 1.81  700 .8117 1.26 16.65 2.40	Sirdir			on		Strain Ar	ea or	
- Oad Oial Strain Area of Kam/cm Pan Rdg  O .5628 O .16.4 O  100 .5535 .211 .16.45 .304  200 .5418 .476 .16.48 .607  300 .5324 .690 .16.52 .908  400 .5309 .724 .16.55 1.21  450 .5295 = .8938	Sirdir			on		Strain Ar	ea or	
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On Rdg Strain Area or kgm/cm Pan Rdg  O .5628 O 16.4 O  100 .5535 .211 16.45 .304  200 .5418 .476 16.48 .607  300 .5324 .690 16.52 .908  400 .5309 .724 16.55 1.21  450 .5295 = .8938 1.36  500 .8908 .824 16.56 1.51  500 .8850 .955 16.58 1.81  100 .8117 1.26 16.65 2.40  200 .8340 2.110 16.78 2.88  100 .8270 2.282 16.80 3.57	Sirdir			on		Strain Ar	ea or	
On Rdg Strain Area of kam/cm Pan Rdg  O .5628 O 16.4 O  100 .5535 .211 16.45 .304  200 .5418 .476 16.48 .607  300 .5324 .690 16.52 .908  400 .5309 .724 16.55 1.21  450 .5295 = .8938 1.36  500 .8908 .824 16.56 1.51  600 .8850 .955 16.58 1.81  700 .8117 1.26 16.62 2.10  800 .8655 1.40 16.65 2.40  700 .8515 1.716 16.65 2.80  100 .8270 2.275 16.80 3.27  1200 .8210 2.282 16.80 3.57	Sirdir			on		Strain Ar	ea or	
- Oad On Rdg Strain Area on Kam/cm Pan Rdg  - O .5628 0 16.4 0  - Rdg Strain Area cm² kam/cm Pan Rdg  - O .5628 0 16.4 0  - Rdg Strain Area cm² kam/cm Pan Rdg  - O .5628 0 16.4 0  - Rdg Strain Area cm² kam/cm Pan Rdg  - Oan Rdg Strain Area cm² kam/cm Pan Rdg  - Oan Rdg Strain Area cm² kam/cm Pan Rdg  - Oan Rdg Strain Area cm² kam/cm Pan Rdg  - Rdg  - Oan Rdg Strain Area cm² kam/cm Pan Rdg  - Rdg  - Oan Rdg Strain Area cm² kam/cm Pan Rdg  - Rdg  - Oan Rdg Strain Area cm² kam/cm Pan Rdg  - Rdg  - Nath Rdg  - Oan Rdg  - Nath Rdg  -	Sirdir			on		Strain Ar	ea or	
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- Oad On Rdg Strain Area on Kam/cm Pan Rdg  - O .5628 0 16.4 0  - Rdg Strain Area cm² kam/cm Pan Rdg  - O .5628 0 16.4 0  - Rdg Strain Area cm² kam/cm Pan Rdg  - O .5628 0 16.4 0  - Rdg Strain Area cm² kam/cm Pan Rdg  - Oan Rdg Strain Area cm² kam/cm Pan Rdg  - Oan Rdg Strain Area cm² kam/cm Pan Rdg  - Oan Rdg Strain Area cm² kam/cm Pan Rdg  - Rdg  - Oan Rdg Strain Area cm² kam/cm Pan Rdg  - Rdg  - Oan Rdg Strain Area cm² kam/cm Pan Rdg  - Rdg  - Oan Rdg Strain Area cm² kam/cm Pan Rdg  - Rdg  - Nath Rdg  - Oan Rdg  - Nath Rdg  -	Sirdir			on		Strain Ar	ea or	
Oad On Rdg Strain Area of Kam/cm Pan Rdg  O .5628 O 16.4 O  100 .5535 .211 16.45 .304  200 .5418 .476 16.48 .607  300 .5324 .690 16.52 .908  400 .5309 .724 16.55 1.21  450 .5295 = .8938 1.36  500 .8908 .824 16.56 1.51  000 .8850 .955 16.58 1.81  700 .8117 1.26 16.62 2.10  800 .8655 1.40 16.65 2.40  700 .8515 1.716 16.67 2.80  1100 .8270 2.275 16.80 3.27  1200 .8210 2.282 16.80 3.57  1200 .8210 2.282 16.80 3.87  1400 .7085 4.97 17.26 4.06  1500 .6868 5.46 17.35 4.32	Sirdir			on		Strain Ar	ea or	
Odd On Rdg Strain Area of Cm Kam/cm Pan Rdg S/6 Cm Kam/cm Kam/cm Pan Rdg S/6 Cm Kam/cm K	Sirdir			on		Strain Ar	ea or	
Oad On Rdg Strain Area of Kam/cm Pan Rdg  O .5628 O 16.4 O  100 .5535 .211 16.45 .304  200 .5418 .476 16.48 .607  300 .5324 .690 16.52 .908  400 .5309 .724 16.55 1.21  450 .5295 = .8938 1.36  500 .8908 .824 16.56 1.51  000 .8850 .955 16.58 1.81  700 .8117 1.26 16.62 2.10  800 .8655 1.40 16.65 2.40  700 .8515 1.716 16.67 2.80  1100 .8270 2.275 16.80 3.27  1200 .8210 2.282 16.80 3.57  1200 .8210 2.282 16.80 3.87  1400 .7085 4.97 17.26 4.06  1500 .6868 5.46 17.35 4.32	.766			on		Strain Ar	ea or	



PROJECT TEST PLE #2 UNIVERSITY of ALBERTA SITE CHEM- PHYSICS BUILDING DEP'T of CIVIL ENGINEERING SAMPLE SOIL MECHANICS LABORATORY LOCATION UNIVERSITY CAMPUS HOLE MOISTURE DEPTH CONTENT P.K. DATE 11/22/58 TECHNICIAN Hole No. 2 Z 71/2 2/2 12 1/2 5' 10 15 Depth Sample No-2 3 1A 130 1A27 Container No-1A27 1AZT 1A35 1A27 97.34 Wt. Sample Wet + Tare 87.95 71.36 67.81 100.64 63.00 Wt.Sample Dry + Tare 11.25 60.36 85.76 62.63 88.83 58.81 16.69 Wt. Water 4.09 11.00 5.18 11.58 11.81 17.67 Tare Container 17.54 17.67 17.66 17.71 17.66 41.15 42.69 44.97 Wt. of Dry Soil 53.71 68.09 71.12 Moisture Content w % 31.2 25.8 16.9 11.6 16.6 10.0 2 Z Hole No 2 20' 17/2 z3' Depth 4 Sample No-Container No-1A19 1AB6 1A23 85.69 67.12 74.41 Wt. Sample Wet + Tare 77.90 6Z.Z9 64.33 Wt. Sample Dry + Tare 4.83 7.79 10.08 Wt. Water 17.38 17.46 17.27 Tare Container 44.91 60.44 47.06 Wt. of Dry Soil 12.9 22.8 Moisture Content w% 10.8 Hole No. Depth Sample No-Container No-Wt Sample Wet + Tare Wt Sample Dry + Tare Wt Water Tare Container Wt. of Dry Soil Moisture Content w% Remarks: \_\_\_

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#### APPENDIX B

## LABORATORY TEST RESULTS

SITE OF WEST-END CITY YARDS



## LABORATORY TEST RESULTS

TEST PILE NO. 3



UNIVERSITY of ALB	TEST PI	LE #3					
SILE THE AUE. E 144						TREET.	
		SAMPLE #1					
SOIL MECHANICS LABORATORY LOCATION  ATTERRERG LIMITS HOLE #3					DEPTH 5'		
ATTERBERG LIMITS HOLE #3 DEPTH 5' TECHNICIAN 7. L. DATEZHI159							
Lia	uid Limit						
Trial No.	2	3	1	Z		3	
No. of Blows 41	41	43	18	19		19	
Container No. V41	V36	¥79	Y71	V4	6	VZO	
Wt. Sample Wet + Tare 84.2053			88.8034	100.48	097 8	0.6532	
Wt· Sample Dry + Tare 77. 7311	75.2533	86.8244	81.0517	92.6	834 7	13.6767	
Wt. Water 6.4742	6.5068	5.8184	7. 7517	7.72	263	6.9765	
Tare Container 69.6448	67.0527	19.5041	72.0433	83.72	237 6	5.5906	
Wt. of Dry Soil 8.0863	8.2006	7.3167	9.008		597	8.0861	
Moisture Content w% 80.1	79.4	79.4	86.2	87.	/	86.4	
Aver	age Values		Plastic	imit			
		Trial No.		1	2	3	
	= 84.2 %	Container	No.	4	7	11	
w	= 24.2 %	Wt. Sample	Wet+Tare	31.7343	31.741	240.6491	
		Wt. Sample	Dry+Tare	31.3717	31.384	1440.3068	
		Wt. Water				8 0.3423	
Ip.	= 60.0%	Tare Cont				439.1062	
87	20.6	Wt of Dry				30 1.40ZI	
	$I_1 = \frac{2.9}{\text{Moisture Content } \%} \frac{24.5}{24.1} \frac{24.4}{24.4}$						
			Shrinkage	Limit			
86	Trial No.					-	
		Container					
85		Wt. Sample					
		Wt Sample	Dry + lare				
2	Wt. Water						
884		Tare Container					
		Wt. of Dry Soil Wo					
500		Moisture Content w%					
0 0 0 0 0		Vol. Container V Vol. Dry Soil Pat Vo					
ŏ		Shrinkage					
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Moisture)		2/	= w (1	Wo X I	00)		
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		firm, n	highly waget s	tructo	repr	esent.	
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UNIVERSITY of ALBERTA PROJECT TEST PILE #3						
DEP'T of CIVIL ENGINEERING SAMPLE #2						
SOIL MECHANIC		ST. & 114th	A			
ATTERBER		RATORY	HOLE	#3		TH 10'
ATTERBERG LIMITS HOLE #3 DEPTH 10' TECHNICIAN Y.K. DATE 1/18/59						
Trial No.	1	2	3	1	2	3
No. of Blows	35	34	33	19	19	18
Container No.	V 79	VZO	V 46	V41	V7/	V36
Wt·Sample Wet + Tare	Vt. Sample Wet + Tare   95.3145   81.3697		98.2450	83.2292	89.7113	81.9485
Wt Sample Dry + Tare	11. Sample Dry + Tare 89.9376 76.0099		93.3234	78.3354		76.5907
Wt· Water	5.3769	5.3598	4.9216	4.8938		5.3578
Tare Container	79.5047	65.5906	83.7237	69.6448	72.0433	67.0527
Wt. of Dry Soil	10.4329	10.4193	9.5997	8.6906	11.3252	9.5380
Moisture Content w%	51.4	51.4	51.4	56.4	55.9	56.2
	Aver	age Values		Plastic L	imit	
	+-+	•	Trial No.		1 2	3
	w	= 53.9 %	Container	No.	4 7	11
	w	= 28.2 %	Wt. Sample	Wet+Tare 3	2.2570 32.5	477 41.6348
	w <sub>s</sub>					57241.0900
	<del>                                      </del>		Wt Water		0.5263 0.5	905 0.5448
	1p	= 25.7 %	Tare Cont	ainer z	9.8903 29.90	01439.1062
57	L * 18.8		Wt. of Dry	Soil	1.8404 2.0:	558 1.9838
	1	= 1.37	Moisture Co	ontent %	28.6 28	.7 27.4
				Shrinkage	Limit	
56			Trial No.			
			Container	No.		
55			Wt. Sample	Net + Tare		
32			Wt Sample [	Dry + Tare		
%		Wt. Water				
54			Tare Container			
-			Wt. of Dry S			
e			Moisture Content w%			
0 U			Vol. Contain			
°			Vol. Dry Soi			
52			Shrinkage \			
			Shrinkage L	imit ws ]		
Moist eration		440	$w_s = w \left( \frac{V - V_o}{W_o} \times 100 \right)$			
w51				C M	10	
Σ			Description	n of Samp	le:	
50					plastic,	firm.
					ove pres	
			mottle	ed grey é	brown.	
			Remarks:			
7 8 9 10 15	20 25	30 40				
7 8 9 10 15 Number	of Blows					

DELLO GE ATREBUIL LIMIVERSITY of ENGLIEELING 11VIS 10 - 193G 3 16 1 5 2 TANTA, MA LABORATORY SOM ME HANGS NACINIDAN ATTERBERG 27 1 1/11/21 arag Light Pink -01/7 I WINE IN 100 d)ner Ma PEV. Apple . gynT : byW slower E-14.38 BIXI 78 0043.8P WASE BY inmove Day - Term ALLELIA YATON ALVE II BISTA 部を込む 説し TILLY E 12 TABLETT AE-C.US 1000 100 내바루 공과 registration a BEAR DY He2 you to 0 HP18.0 15 44 33 BALVESSE. the instance stall STATE STATES PROPERTY ADDRESSAY Trial No. FIRE YUP and sprinters. 11-50m wie Weltting 35 25 50 mod 11 We Some in Dirt a Toronal Constitution of the 174 with a limite pour s SALDW AM Toris Correiner a 81 - 1 Who hory Soil - S.S. W. Theling emilans TEN = 1 Shrinkess Limit -oM Indi Containar Mo. BINT THE WAT THEIR Wh Sommia Day, y Turn 16709 199 าแกได้สักษา 5101 W DIS VIC TO TW Monthly & Contrast Act to V sentetend dev AV 109 ROZ WG TOV W-V IN SOMMING pw omil top ognite (00) x - 1 - 1 - 100) religioned to noticisperu. ments and the second se Ham ark to d) 01 8 B OF amole to takmuli

UNIVERSITY of ALBERTA PROJECT TEST PILE #3 SITE 114th Ase. \$ 144th STREET.								
DEP'T of CIVIL ENGINEERING SAMPLE #3								
	SOIL MECHANICS LABORATORY LOCATION							
ATTERBERG LIMITS HOLE #3 DEPTH 15'. TECHNICIAN P. K. DATE 1/18/59								
		uid Limit	TECHNI	CIAN P.	K.	DATE /	/18/59	
Trial No.		2	3	1	7		3	
No. of Blows 29		26	29	16	17		18	
Container No. V41		V19	V36	V20	V 71	•	V 46	
	· Sample Wet + Tare 86.6003 96.4950		83.6643				.9474	
Wt. Sample Dry + Tare 79.98								
	143							
Tare Container 69.6	448	79.5047						
Wt. of Dry Soil 10.3	412	10.3469	10.1458	10.4210	10.73	334	9.6974	
Moisture Content w% 63	.9	64.1	63.7	67.4	66.	9	67.3	
	Aver	ago Valuos		Plastic I	imit			
	H	age Values	Trial No.		1	2	3	
68	w	2= 64.7%	Container	No.	4	7	11	
	w	30.2 %	Wt. Sample	Wet+Tare	33.8988	33,3584	A42.0207	
	ws	r	Wt. Sample	Dry + Tare	32.9590	37.5561	4-1.3512	
	# Ws	31159	Wt. Water		0.9398	0.802	3 0.6695	
ΛΦ1	1p	= 34.5 %	Tare Cont				A39.106Z	
67	I I	<b>15.7</b>	Wt. of Dry				7 Z.Z450	
	Ħ T.	= 2.2	Moisture C	ontent %	30.6	30.2	Z9.8	
	∄ ~'	***************************************		Shrinkage	Limit			
			Trial No.					
			Container	No.				
	Wt. Sample Wet + Tare							
66	Wt. Sample Dry + Tare							
Wt. Water								
8	Tare Container							
	Wt. of Dry Soil Wo							
e e	Moisture Content w%							
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63								
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			il					
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PROJECT UM ERSITY OF ALBERTA DEP T. of CIVIL ENGINEERING 3 91105 SOIL MECHANICS MOLTAGORA LAIORATORY - 41930 EIMITS ATTERBERG BUOH 31A0 FUTHICIAN tia d biupid NO. Two 3 10 LONGE NO. PILE Somple Wert Tore | en ecos O-S-BILLIANS THE RICHTS ARE DUTTED IN BUILDING ILIBIE BEEFE anoth year stamps , , BANA THE BOW anta an Itado to 540 Sell 1 Avenue Pol 3P0101005 | 41 F F (42 EL.O) (11 14) 51 4 34 10 1 1305 OL 314E SI Hos Byd No PARTY CONTRACTOR - 10 V 10 730 Planta Dillaria Avarous Values mw tpiny 5 1 + 2 27W ·oMaskijohuba Wy Sample Widthard U.S. STREET, 20 St. Male of and the second of the part of the part of the second of th Find trans arrest metew aw TO 45 . 2 01 Torre Container Levinos Factorios ever Mareture Content St. Jens Jones 6 Com 44.00 Time object of -DM - DM -pW sanipland Wr Sample Wet + Tare Wi Semple Dry + Tore 28/20/W 138 Tara Container W. ME BEY SON W. Streeture Centruit 4075 Vol. Cowlanne V VOV DEF SON POT VA W-V GOV DODANIANE Shrinkoga Limit Wa (00) = -1-1 back to noity were Ramgraus GH-XAG Number of Blank

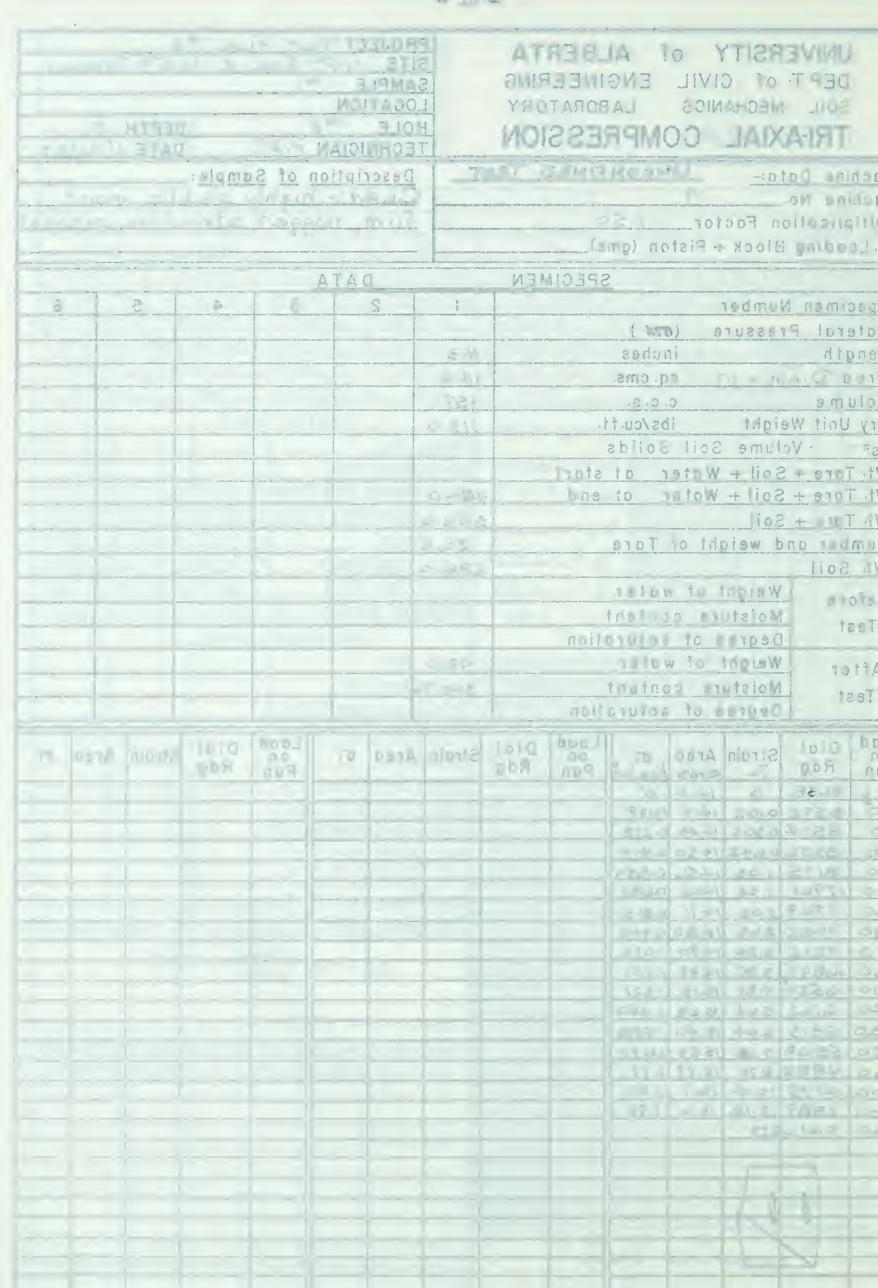
UNIVERSITY of	ALB	ERTA	PROJEC	TEST T	E 144	3	
DEP'T of CIVIL EI	NGIN	EERING	SAMPLE	· · · · · · · · · · · · · · · · · · ·	१ । ५५	STR	EET.
		RATORY	LOCATIO				
ATTERBERG		MITS	HOLE	#3		DEPTH	20'
ATTENDENO	<u></u>	IVITO	TECHNI	CIAN P.	K.	DATE	1/19/59
	Liq	uid Limit					
Trial No.		Z	3	1	Z		3
No. of Blows 27		26	25	/3	14		15
Container No. V36		V.41	VZO	V46	V79	7	V 71
Wt. Sample Wet + Tare 87.14	72	86.8683	85.7216	102.6605	94.85	41 89	9.2773
Wt. Sample Dry + Tare 81.11	49	81.7073	79.6938	96.6626			3.8578
Wt. Water 6.03		5.1610	6.0338				5.4195
Tare Container 67.05		69.6448					2.0433
Wt. of Dry Soil 14.00		12.0625	14.1232				1.8145
Moisture Content w% 42	7	4-2.7	4-2.7	46.3	46.1		45.8
	Aver	age Values		Plastic	Limit	<del></del>	
	Н	= 42.8 %	Trial No.		1	2	3
47			Container	No.	4	7	11
	w	23.8 %	Wt. Sample	Wett lare	34.35 //	33.925	1 121/270
	Ws	s	Wt. Sample	DIX T lare			2 0.7970
		= 19.0%	Wt. Water Tare Cont	giper			439.1062
	77		Wt of Dry				0 3.3216
46	11	* /Z-7	Moisture C				
	It	= 1.5		Shrinkage			
	H.,,,,,,,		Trial No.	Jiii iiikugu			
			Container	No.		4 **	
			Wt Sample				
45			Wt. Sample				
			Wt. Water				
8			Tare Cont	ainer			
			Wt. of Dry				
Content			Moisture Co	ntent w%			
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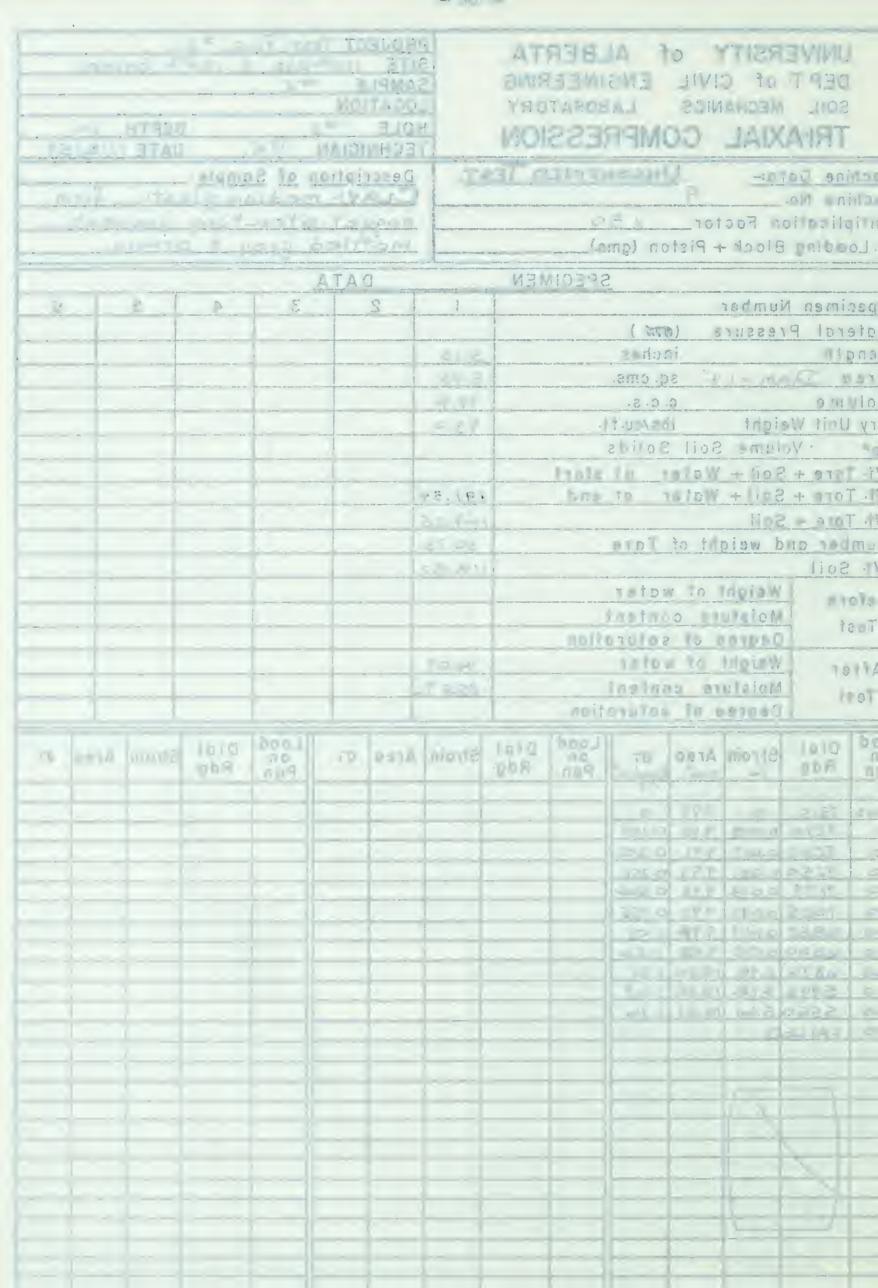
UNIVERSITY of	ALE	BERTA	PROJEC	TEST :	PILE #	3	
		EERING	SITE	14th Ave	= = 14	4+4 5	STREET.
		RATORY	LOCATION				
ATTERBERG		MITS	HOLE	#3		DEDT	H 25'
ATTEMBLIG		IVIII	TECHNI	CIAN P.			1/20/59
	Liq	uid Limit					-1-7-1
Trial No.		2	3	1	Z		3
No. of Blows 3°	7	40	40	17	18		19
Container No. V7		V41	V36	V79	V2.	0	V46
	089	93.7733	84.9006	97.3150	84.10	JOZ /	100.7605
1010 101	3375	88.1272	80.6900	92.9273	79.4		96.550
	7714	5.6461	4.2106	4.3877		259	4.210
1414	0433	69.6448	67.0527	79.5047			83.723
	2942	18.4824	13.5373	13.4220			12.826
Moistare Convent W/s <sub>1</sub> 30	7.6	30.5	31.1	<b>3</b> 2.6	33.	1	32.8
	Aver	age Values		Plastic	Limit		
34	++1	= 32.0 %	Trial No.		1	2	3
34	3 1 1		Container		4	7	//
	111		Wt. Sample				
	$  w_s  $	2	Wt Sample Wt Water	DINTIGIE			
	In In	= 15.6 %	Tare Cont	miner			0Z 0.538 1439.106
33	14		Wt. of Dry				90 3.295
33			Moisture Co			16.1	
	11 11	= 2.42		Shrinkage			
	North		Trial No.				
			Container I	Vo.			
32			Wt-Sample V				
32			Wt Sample D				
%			Wt Water				
			Tare Conto				
+			Wt. of Dry S				
			Moisture Co				
0			Vol. Contair Vol. Dry Soi				
o line			Shrinkage V				
			Shrinkage V Shrinkage L				
W So Triangle of the second of							
7 30			Ws	= w (Y	-Vo x 10	00)	
Ö					100		
Σ			Description	n of Sam	ple:		
			GLACIA			e, are	zy, sand
			coaltp	rea grav	rel Pre	sent	mediun
			plastic				
			Remarks:_				
7 8 9 10 15 2 Number of	0 25 Blows	30 40					
Mamper of	DIOMS						

TOBLORA UNIVERSITY OF ALBERTA DEET of GAME ENGINEERING SAMPLE TROTAROBAL MOIT POLE MECHANNES - 117930 ERBERG LIMITS BTAG LIGHT PINCE 1034 Swole ! TE-SUL D old senis 100 100 ELVITOR engt Haw sigms 8949-89 augus Beg + Thus THEOR 725m4 6013.0 91394 April 16 1.4.12 F BARATO STAR III. ISSOCIAL. Maniphood . 13 5-8 £1 THE SOLI lure Content with an in - WAL rimui pirep 19 AVERGOD VOIDES coll\_loisT BOL THE ontainer Ma-3 4 4 50 We Sample Well late We Somply Dig Tore THIRW IW THE PLAN PRINCIPLE Torse Containing ACTUAL TO SEED AND ASSESSED. Use you to W 1 - 1 Lie inginad grutaink 147 20 SAMAROOM WAREI J -10M ROSET -cM nemistred MILET ET WILL TOUR WI Semple Dry + Term w.tnW-W Sanialman Carrings W Not all to My FOR Inches Controls. W ventokneti -tev Vol Diy Sell Pel . W N-V lov apolining Strinkoge Limit wa (001 = N-V) -0 = DI paseription of Someter CARLING SUPPLIES AND ARREST the party of the p Ramorkan CE 20 05 40 0198 Number of Stave

	UNIV	ERSIT	Y	of Al	LBEF	RTA		PROJ	ECT	THAJE	ILE =	3		
	DEP '	Γ· of	CIVIL	ENG	INEE	RING		SAME	LE	#1	. ξ /	44.~	STRE	ET
	SOIL	MECH	ANICS	LA	BORAT	TORY			TION					
				OMPF				HOLE		3		DEPTH	5'	
										NP.K.		DATE		159
Mach	ine D	ata:-	U	HCOH	FINE:	DTE	ST.			on of S	7		1 1	
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Load on Pan	Dial Rdg	Moist Degree Strain	a Area cms.	ontent satura kgm/cm	Load		34.3	70	5.	on	1	Strain	Area	51
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Load on Pan	Dial Rdg .8638 .859.	Moist Degree Strain 0 0.105 4 0.305 6 0.642	ure c e of Area cms. 14.4 14.45 14.45	kam/cm 0 0 0 0 0 0 0 0 0 0 0 0 0	Load		34.3	70	0 5.	on	1	Strain	Area	σι
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Load on Pan O 9 40 80 1/20 1/60 200	Dial Rdg .859. .859. .835 .817:	Moist Degree Strain 0 0.105 40.305 60.642 51.03	ure c e of Area cms. 14.4 14.45 14.45 14.50 14.50	ontent satura Kgm/cm o 0.278 0.414 0.550 0.682	Load		34.3	70	57.	on	1	Strain	Area	51
Te: Load on Pan 0 9 40 80 120 160 240	Dial Rdg .8638 .859 .850 .835 .8173	Moist Degree Strain 5 0 0 0.105 4 0.305 6 0.642 5 1.03 1.52 9 2.02	ure c e of Area cms. 14.4 14.45 14.45 14.50 14.50 14.63	cisq c.414 c.550 c.815	Load		34.3	70	5.	on	1	Strain	Area	61
Te: Load on Pan 0 9 40 80 120 160 240 240 280	Dial Rdg .8638 .859 .859 .835 .8173 .796 .774	Moist Degree  Strain  0  0.105  0.305  0.642  1.52  7.02  2.03	ure c e of Area cms. 14.4 14.45 14.45 14.56 14.56 14.63 14.71 14.80	ontent satura Kgm/cm o 0.139 0.278 0.414 0.550 0.682 0.815 0.945	Load		34.3	70	0 5.	on	1	Strain	Area	σι
Te: Load on Pan 0 9 40 80 120 160 240 240 280 320 360	Dial Rdg .8638 .859 .859 .835 .817; .796 .774 .748	Moist Degree  Strain  0  0.105  0.305  0.642  1.52  7.02  2.02  2.63  2.3.24  2.3.95	ure c e of Area cms. 14.4 14.45 14.50 14.50 14.50 14.63 14.71 14.80 14.90	ontent satura kgm/cm o 0.139 0.278 0.414 0.550 0.682 0.815 0.945 1.013 1.201	Load		34.3	70	0 5.	on	1	Strain	Area	51
Te: Load on Pan 0 9 40 80 120 160 240 280 320 340 400	Dial Rdg .8638 .859 .859 .835 .817 .796 .774 .748 .721 .689	Moist Degree  Strain  0  0.105  40.305  60.642  51.03  11.52  72.02  72.03  73.74  74.81	ure c e of Area cyns. 14.4 14.45 14.50 14.50 14.63 14.11 14.80 14.90 14.99 15.13	0ntent satura Kgm/cm 0139 0.278 0.414 0.550 0.682 0.815 0.945 1.013 1.201 1.321	Load		34.3	70	5.	on	1	Strain	Area	51
Te: Load on Pan 0 9 40 80 120 160 240 240 280 320 360 400 440	Dial Rdg .8638 .859 .859 .835 .817: .796 .774 .748 .721. .689 .652	Moist Degree  Strain  0  0 0.105  40.305  60.642  51.03  1.52  72.02  2.03  2.03  2.3.24  2.3.95  4.81  2.5.64	ure c e of Area cyns. 14.4 14.45 14.50 14.50 14.63 14.11 14.80 14.90 14.90 14.99 15.13 16.28	07. kgm/cm 00.278 0.414 0.550 0.682 0.945 1.013 1.201 1.321 1.440	Load		34.3	70	0 5.	on	1	Strain	Area	51
Te: Load on Pan 0 9 40 80 120 160 240 280 320 360 400 440 480	Dial Rdg .8638 .859. .859. .835 .817: .796 .774 .748 .721. .689 .652	Moist Degree  Strain  0  0.105  40.305  60.642  51.03  1.52  72.02  2.03  2.3.24  2.3.95  4.81  2.5.64	ure c e of Area cms. 14.4 14.45 14.50 14.50 14.50 14.63 14.71 14.80 14.90 14.99 15.13 15.28 15.40	0ntent satura Kam/cm 0 0.139 0.278 0.414 0.550 0.682 0.815 0.945 1.013 1.201 1.321 1.440 1.558	Load		34.3	70	7 5.	on	1	Strain	Area	σ,
Te:  Load on Pan 0 9 40 80 120 160 240 240 280 320 400 440 480 520	Dial Rdg .8638 .859 .859 .859 .835 .817: .796 .774 .748 .721. .689 .652 .616 .580	Moist Degree  Strain  0  0 0.105  40.305  60.642  51.03  1.52  72.02  2.03  2.03  2.3.24  2.3.95  4.81  2.5.64	ure c e of Area cyns. 14.4 14.45 14.50 14.50 14.63 14.71 14.80 14.90 14.99 15.13 15.28 15.40 15.53	0ntent satura Kgm/cm 0' 0139 0.278 0.414 0.550 0.682 0.815 0.945 1.013 1.201 1.321 1.440 1.558 1.672	Load		34.3	70		on	1	Strain	Area	51
Te:  Load on Pan 0 9 40 80 120 160 240 280 320 360 400 440 520 560 600	Dial Rdg .8638 .859 .859 .859 .817: .774 .7748 .721 .689 .652 .616 .580 .488 .417:	Moist Degree  Strain  0  0.105  40.305  60.642  51.03  11.52  72.02  22.63  23.24  23.95  44.81  25.64  5.64  7.28  28.74  510.4	ure c e of Area cyns. 14.4 14.45 14.50 14.50 14.63 14.71 14.80 14.90 14.90 14.99 15.13 15.28 15.28 15.79 16.1	0ntent satura Kgm/cm 0/139 0.278 0.414 0.550 0.682 0.815 0.945 1.013 1.201 1.321 1.440 1.558 1.672 1.77 1.86	Load		34.3	70	57	on	1	Strain	Area	51
Te: Load on Pan 0 9 40 80 120 160 240 280 320 340 440 440 480 520 560 600 640	Dial Rdg .8638 .859 .859 .835 .817; .796 .774 .748 .721, .689 .652 .616 .550 .488 .417; .288	Moist Degree  Strain	ure c e of Area cyns. 14.4 14.45 14.50 14.50 14.63 14.71 14.80 14.90 14.90 14.99 15.13 15.28 15.28 15.79 16.1	ontent satura kgm/cm o 0.139 0.278 0.414 0.550 0.682 0.815 0.945 1.013 1.201 1.321 1.440 1.558 1.672 1.77	Load		34.3	70		on	1	Strain	Area	51
Te:  Load on Pan 0 9 40 80 120 160 240 280 320 360 400 440 520 560 600	Dial Rdg .8638 .859 .859 .835 .817; .796 .774 .748 .721, .689 .652 .616 .550 .488 .417; .288	Moist Degree  Strain	ure c e of Area cyns. 14.4 14.45 14.50 14.50 14.63 14.71 14.80 14.90 14.90 14.99 15.13 15.28 15.28 15.79 16.1	0ntent satura Kgm/cm 0/139 0.278 0.414 0.550 0.682 0.815 0.945 1.013 1.201 1.321 1.440 1.558 1.672 1.77 1.86	Load		34.3	70		on	1	Strain	Area	61
Te: Load on Pan 0 9 40 80 120 160 240 280 320 340 440 440 480 520 560 600 640	Dial Rdg .8638 .859 .859 .835 .817; .796 .774 .748 .721, .689 .652 .616 .550 .488 .417; .288	Moist Degree  Strain	ure c e of Area cyns. 14.4 14.45 14.50 14.50 14.63 14.71 14.80 14.90 14.90 14.99 15.13 15.28 15.28 15.79 16.1	0ntent satura Kgm/cm 0/139 0.278 0.414 0.550 0.682 0.815 0.945 1.013 1.201 1.321 1.440 1.558 1.672 1.77 1.86	Load		34.3	70		on	1	Strain	Area	51
Te: Load on Pan 0 9 40 80 120 160 240 280 320 340 440 440 480 520 560 600 640	Dial Rdg .8638 .859 .859 .835 .817; .796 .774 .748 .721, .689 .652 .616 .550 .488 .417; .288	Moist Degree  Strain	ure c e of Area cyns. 14.4 14.45 14.50 14.50 14.63 14.71 14.80 14.90 14.90 14.99 15.13 15.28 15.28 15.79 16.1	0ntent satura Kgm/cm 0/139 0.278 0.414 0.550 0.682 0.815 0.945 1.013 1.201 1.321 1.440 1.558 1.672 1.77 1.86	Load		34.3	70		on	1	Strain	Area	51
Te: Load on Pan 0 9 40 80 120 160 240 280 320 340 440 440 480 520 560 600 640	Dial Rdg .8638 .859 .859 .835 .817; .796 .774 .748 .721, .689 .652 .616 .550 .488 .417; .288	Moist Degree  Strain	ure c e of Area cyns. 14.4 14.45 14.50 14.50 14.63 14.71 14.80 14.90 14.90 14.99 15.13 15.28 15.28 15.79 16.1	0ntent satura Kgm/cm 0/139 0.278 0.414 0.550 0.682 0.815 0.945 1.013 1.201 1.321 1.440 1.558 1.672 1.77 1.86	Load		34.3	70		on	1	Strain	Area	σι
Te: Load on Pan 0 9 40 80 120 160 240 280 320 340 440 440 480 520 560 600 640	Dial Rdg .8638 .859 .859 .835 .817; .796 .774 .748 .721, .689 .652 .616 .550 .488 .417; .288	Moist Degree  Strain	ure c e of Area cyns. 14.4 14.45 14.50 14.50 14.63 14.71 14.80 14.90 14.90 14.99 15.13 15.28 15.28 15.79 16.1	0ntent satura Kgm/cm 0/139 0.278 0.414 0.550 0.682 0.815 0.945 1.013 1.201 1.321 1.440 1.558 1.672 1.77 1.86	Load		34.3	70		on	1	Strain	Area	61
Te: Load on Pan 0 9 40 80 120 160 240 280 320 340 440 440 480 520 560 600 640	Dial Rdg .8638 .859 .859 .835 .817; .796 .774 .748 .721, .689 .652 .616 .550 .488 .417; .288	Moist Degree  Strain	ure c e of Area cyns. 14.4 14.45 14.50 14.50 14.50 14.63 14.71 14.80 14.90 14.90 14.99 15.13 15.28 15.28 15.79 16.1	0ntent satura Kgm/cm 0/139 0.278 0.414 0.550 0.682 0.815 0.945 1.013 1.201 1.321 1.440 1.558 1.672 1.77 1.86	Load		34.3	70		on	1	Strain	Area	51



	UNIVE										ST PIL		th ST	0000	
	DEP T							ŞAM		-	# 2	7 144	. 27	KEE!	
	SOIL	MECHA	NICS	LA	BORAT	ORY		LOC		ON					
	TRI-A	ΧΙΔΙ	CC	MPR	FSS	ION		HOL	E	#3			DEPTH	10	
								TEC	HNI	CIAN	P.K		DATE	/18/	59
	ine Dat			JUCON	IFINE	DTE	SI.	De	scr	iptior	of S	ample:			
	ne No.		_					C	A	4 - Y	nedic	m pla	istic	fir	m,
	olicatio							no	99	et:	struc	tore	pres	sent	
Wt. Lo	ading 1	Block ·	+ Pist	on (gm	s.)			m	ot	tled	gre	y E	marc	۸	
					SPECII	MEN				ATA					
	imen N						1		2		3	4	5		6
	ral Pi	ressur													
Lenc				nches	·		3.15								
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	m e						79.4								
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	Tare +														
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	ber an	d weig	ght of	Tare			30.7							_	
W1.	Soil						118.5	Z					-		
Befo				water											
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Aft		Weight				-	42.0	7					-		
Tes				ontent			35.5	79							
		Degre	e of	satura	tion										
Load	Dial	C+:-	A = 0 =		Load	Dial	Strain	Λ		5.	Load	Dial	Strain	0.00	51
on Pan	Rdg	Strain	Cm	Kgm/cm	on Pan	Rdg		Are	u	01	Pan	Rdg	Siruiii	Aled	01
				1/											
Ogms	.7315	0	9.93					1	-						
26	.7290								-						
40	.7262										-				
50	.7250						-								
150	.7025														
200	.6852														
250	.6500	0.575	9.98	1.26											
300	.6376	2.98	10.20	1.51											
350	.5995	,		1											
370	.5550 FAILE	1	10.51	1.10											
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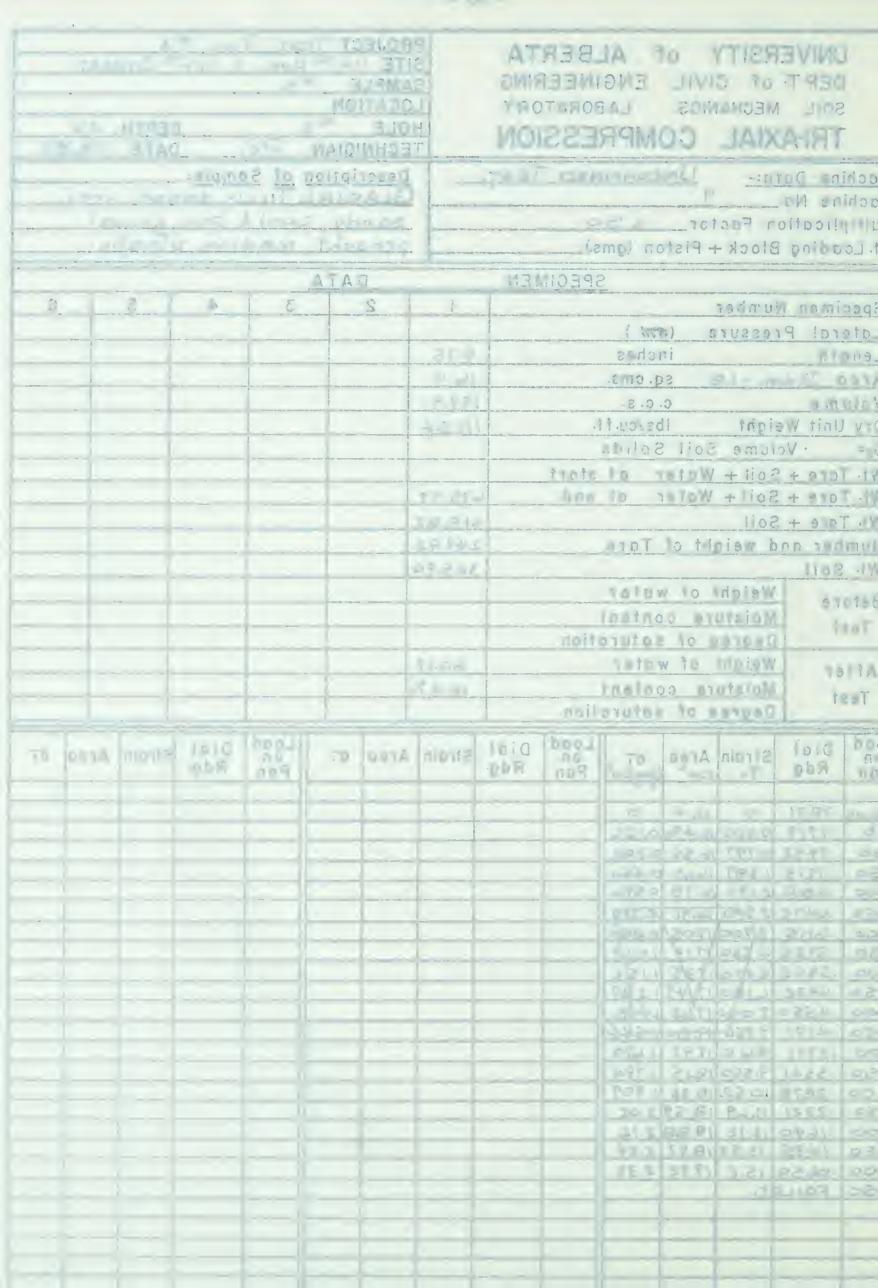
	UNIVE	RSIT	Y o	f Al	BEF	RTA		PROJI	ECT TE	ST PI	LE #	3		
				ENG						AYE	£ 141	tth S	TREE	T.
				LA				LOCAT		# 3				
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	IRIA	XIAL	C	OMPR	RESS	NOIS		TECHI	VICIAN	J P.K	,	DEPTH DATE		50
Mach	ine Da	t a:-	1	Jucon	LEINE	DTE	ST	-					1/18/	37
Mach	ine No.		q	i		- 15		Cin	TIPITO	n of S	ambie:	اد اد،	c = +. =	
				× 50	)			Mo		7,1	11.11. 1	igh Pi	asiic	,
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					SPECI	MEN		_	DATA					
	imen 1						1	- 2		3	4	5		6
	ral P	ressur		07h )										
Len				nches			3.6							
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				bs/cu-f			87.8							
				Solids										
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			Wate	r at	end		320.6	5						
	Tare +						244.13	3						
	ber an	d weigh	ght of	Tare			30.7				- <del></del>			
Wt	Soil			,			213.4	0						
Befo	re			water										
Tes	+			ontent										
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Aft	er	Weight	t of v	water			76.52	7						
Te	st	Moistu	ire c	ontent			35.9	10						
		Degre	e of	satura	tion									
Load	Dial				Load	Dial				Load	Dial			
on Pan	Rdg	Strain	Area	01	on Pan	Rdg	1.511 (111)	Area	5	Pan	Rdg	Strain	Area	51
		1-1-	CIA	Kgm/cm										
ogms 50	.BZOO		16.4	0										
	.8096													
100	.1910								-	1				
200	.7700	-	1											
250	.7304													
300				D.888										
350	.6880		17.01							-				
400	.6650		17.15					-	-					
450		4.97 6.05		1.30										
500 550	FAILE		11.73	1.43										
		17												
		1/												
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Tolobase 3 UNIVERSITY OF ALBERTA DE FOR DVIL ENGINEERING THE MECHANICS LABORATORY HOLE TECHNICIEM ONLE TRIAMIAL COMPRESSION TATIL OF LIST HOUSE Description & Salvan. achine Dataachine No. and the transfer and the ultiplication Factor Loading Block + Piston (ums.) SPECIMEN DATA h 8 73 1 Sp cim, n Number dieral Pressure (67%) .ength Inches rec sa cms. Sede e mulo) C. C S. ry Unit Weight Ibs/cu.Tl. ~ 's= Volume Soll Solids VI Tare + Soil + Water at start Vi-Ture - Soit Water at and Vi- Turn - Soil 6123 unage and weight of Tare lice V Weigh of wole sefore Moisty, e Baniani Test Dearse of saluration Weight of wutter garrawana 1 Tetta Mointure content ISST Degr. a of Lature len Loga boal DD O'd Strain Area III 10123 lolo Strain Ateo) or 79 ONTA NIONE Rdo nb8 ptin 409 71 n. One più Opinio illi DOST NOW BEEN BUTT 200 24 111100 total real real real 0 0 3 30 45 TO LOT JULY BE 20170 16 58 -COLUMN TO THE BOTH BOTH BY CONTRACTOR PORT AND ADDRESS OF

	UNIVE	RSIT	Yo	f A	LBEF	RTA		PROJ	ECT	EST P	LE #	3		
	DEP T							SITE	114.11	AVE	£ 144°	Th STR	REET.	
	SOIL							SAMP		井十				
								LOCA	TION	3				
	TRI-A	XIAL	C(	<b>JMPF</b>	RESS	SION		HOLE				DEPTH		
Mach	ino Da	4	11	.10		7				V P.K		DATE	1/18/	59
	ine Da			NCON	FINED	IES-		Des	criptio	on of S	ample:		-	
	ine No							CLE	Y	stiff, I	righly	Plas	tic, g	rey,
	plicatio							me	sist,		L I SA			
Wt. Lo	ading	Block	+ Pist	on (gr	ns.)			-	11.0					
					SPECI	MEN			DATA	٨				
Spec	cimen N	dum he	r	_	OI LOI	WEIN		1				T =	-	_
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	eral P	ressur		57H )			,							
	gth			nches			4.0							
	O DIA	M = 1.8					16.4							
	ı m e			C. C. S.	-		166.5							
	Unit We			bs/cu-f			85.0	>						
G <sub>S</sub> =	· Vo	lume	Soil	Solids	3							l.		
Wt.	Tare +	Soil +	Wate	r at	start									
Wt.	Tare +	Soil +	Wate	r at	end		339.18	2			-			
Wt.	Tare +	Soil					258.0	3						
	ber an		aht of	Tare			30.7							
	Soil			<u> </u>			227.2							
		Weigh	t of	water	·		221.2							
Befo				ontent								-		
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		Degre	6 01	saturo	ation								_	
		144 - 1 - 4							1					
Aft	0.	Weigh				-	81.1:							
		Moistu	ire c	ontent		-	81.1: 35.7							
Aft Te:		Moistu	ire c											
Te	st	Moistu Degre	e of	ontent satura	ation	0:-1	35.7	%		III oad	0:-1			
Te	Dial	Moistu Degre Strain	e of	satura	Load	Dial	35.7		5.	Load	Dial	Strain	Area	51
	st	Moistu Degre	e of	ontent satura	Load	Dial Rdg	35.7	%	5.	Load on Pan	Dial Rdg.	Strain	Area	51
Te: Load on Pan	Dial Rdg	Moistu Degre Strain	e of Area	ontent satura or z Kgm/cm	Load		35.7	%	5.	on		Strain	Area	51
Te: Load on Pan	Dial Rdg	Moistu Degre Strain	Area cm	ontent saturo Kgm/cm	Load		35.7	%	5.	on		Strain	Area	51
Load on Pan	Dial Rdg -7642 -6843	Moistu Degre Strain	Area cm	ontent satura or kym/cm	Load		35.7	%	5.	on		Strain	Area	51
Load on Pan	Dial Rdg -7642 -6843	Moistu Degre Strain	Area cm 16.4 16.73	ontent saturo Kgm/cm	Load		35.7	%	5,	on		Strain	Area	5.
Load on Pan	Dial Rdg -7642 .6843 .6053	Strain  2.00  3.97  5.67	Area cm 16.4 16.73 17.08	ontent saturo Kgm/cm 0.15 0.29 0.43	Load		35.7	%	5.	on		Strain	Area	61
Load on Pan	Dial Rdg -7642 .6843 .6053 .5375	Strain 2.00 3.97 5.67 7.55	Area cm 16.4 16.73 17.08 17.40	ontent saturo Kgm/cm 0.15 0.29 0.43 0.56	Load		35.7	%	5,	on		Strain	Area	5.
Load on Pan	Dial Rdg -7642 -6843 -6053 -5375 -4622	Strain  Strain	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	57,	on		Strain	Area	51
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	5,	on		Strain	Area	51
Load on Pan	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	5,	on		Strain	Area	57
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	57.	on		Strain	Area	51
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	5,	on		Strain	Area	5.
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	57.	on		Strain	Area	61
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	57,	on		Strain	Area	57
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	57,	on		Strain	Area	57
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	5,	on		Strain	Area	61
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	5,	on		Strain	Area	57
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	5.	on		Strain	Area	61
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	5,	on		Strain	Area	57
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	67	on		Strain	Area	61
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07. 26/24/24/24/24/24/24/24/24/24/24/24/24/24/	Load		35.7	%	57,	on		Strain	Area	57
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07.2 Kgm/cm 0.15 0.29 0.43 0.56 0.69	Load		35.7	%	57,	on		Strain	Area	51
Te: Load on Pan 50 100 150 250 300	Dial Rdg -7642 .6843 .6053 .5375 .4622 .3721 .1400	Moistu Degre Strain % 2.00 3.97 5.67 7.55 9.82	Area cm 16.4 16.73 17.08 17.40 17.18 18.2	07.2 Kgm/cm 0.15 0.29 0.43 0.56 0.69	Load		35.7	%	67,	on		Strain	Area	5.

UNIVERSITY OF ALBERTA SAMP! DERT OF CIVIL ENGINEERING SCIL VECHATICS I BOPATORY LUMAIL 1 1 9 THE JAL COMPRESSION FQ1 = TECHNICIAN \_\_\_\_\_DNTF TREE GLINSPENSEL -ntoG\_unlicol Description 2: - one u. ultiplication Factor 1. Loading Block + Piston (gine) SPECIMEN 4140 Specimen 'umber 9 1 0 oteral Pressur, (and) encth asroni 0.8 9 ... 8101 2.0 2 ory Unit Weight Iberewit. \_ = • Se Volume Soil Solids the love + Soil + Water at there Mr. Tara + Soll + Water at end LIDE + OTHER WAY timbe and miant of ta Miga .M \$3.533 W ight of wole a cotor merann aution Taet Degree of soluration had the ci water \_ 1 / ... TSTEA Mais ur confert P D DE T.st חפנים ס' גמועים וחו בורְמוֹח ברפת מד 0.0 5001 Didi Lings 1010 1000 THE STREET MINIS DE LO MONTE 109 259 AX P.C.O. -02/7 009 Name of the same o the state of the s 12 0 St 11 32 1 2 1 F. . 4 91 192 1 15 1 0

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Tes	st	Moist	re c e of	ontent satura		Dial Rdg	Strain	%	rea	57.	Load on Pan	Dial Rdg.	Strain	Area	51
Load on Pan	Dial Rdg	Moistu Degre Strain	e of Area	ontent satura Kgm/cm²	Load		Strain	%	rea	5.	Load on Pan	Dial Rdg.	Strain	Area	51
Load on Pan	Dial Rdg	Moistu Degre Strain	Area cm	ontent satura Kgm/cm	Load		Strain	%	rea	5.	Load on Pan	Dial Rdg.	Strain	Area	61
Load on Pan	Dial Rdg -7871 -7719	Moistu Degre Strain	Area cm 16.4	ontent satura Kgm/cm <sup>2</sup> O.152	Load		Strain	%	rea	57.	Load on Pan	Dial Rdg.	Strain	Area	67
Load on Pan	Dial Rdg -7871 -7719 -7492	Moistu Degre Strain % 0.320 0.797	Area cm² 16.4 16.45	ontent satura Kgm/cm <sup>2</sup> O.152 0.302	Load		Strain	%	rea	5.	Load on Pan	Dial Rdg.	Strain	Area	57
Load on Pan	Dial Rdg -7871 -7719 -7492 -7213	Moistu Degre Strain % 0.320 0.797	Area cm 16.4 16.55 16.63	ontent satura Kgm/cm <sup>2</sup> 0.152 0.302 0.450	Load		Strain	%	rea	57.	Load on Pan	Dial Rdg.	Strain	Area	67
Load on Pan	Dial Rdg -7871 -7719 -7492 -7213 -6841	Strain 9 0.320 0.797 1.387	16.4 16.4 16.55 16.63	ontent satura Kgm/cm 0 0,152 0.302 0.450 0.596	Load		Strain	%	rea	5.	Load	Dial Rdg.	Strain	Area	61
Te: Load on Pan  Oams 50 150 250 250 300	Dial Rdg .7871 .7719 .7492 .7213 .6841 .6475	Moistu Degre Strain % 0.320 0.797 1.381 2.170 2.940 3.700	16.4 16.4 16.55 16.63 16.78 16.91	0 ntent satura (7 / 2 / 2 / 2 / 2 / 2 / 2 / 2 / 2 / 2 /	Load		Strain	%	rea	57.	Load	Dial Rdg.	Strain	Area	67
Te: Load on Pan  Ooms 50 150 250 250 350 350	Dial Rdg -7871 -7719 -7492 -7213 -6841 -6475 -6115	Moistu Degre Strain % 0.320 0.797 1.381 2.170 2.940 3.700 4.630	16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/	0 ntent satura K9m/cm 0 0.152 0.302 0.596 0.596 0.880 1.018	Load		Strain	%	rea	57.	Load	Dial Rdg.	Strain	Area	67
Te: Load on Pan  Oams 50 100 150 250 350 400	Dial Rdg -7871 -7719 -7492 -7213 -6841 -6475 -6115 -5125 -5345	Moistu Degre Strain % 0.320 0.797 1.381 2.170 2.940 3.700 4.630 5.320	16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/ 17.19	0 ntent satura (9m/cm <sup>2</sup> ) 0.152 0.302 0.596 0.596 0.880 1.017 1.152	Load		Strain	%	rea	57.	Load	Dial Rdg·	Strain	Area	57
Te: Load on Pan  Ooms 50 150 250 250 350 400 450	Dial Rdg -7871 -7719 -7492 -7213 -6841 -6475 -6115 -5725 -5345 -4932	Moistu Degre Strain % 0.320 0.797 1.387 2.170 2.940 3.700 4.630 5.320 6.180	16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/ 17.19 17.35 17.49	0 ntent satura K9m/cm 0 0.152 0.302 0.596 0.596 0.880 1.018 1.152 1.287	Load		Strain	%	rea	51	Load	Dial Rdg.	Strain	Area	57
Te: Load on Pan  Ooms 50 100 250 250 350 400 450 500	Dial Rdg -7871 -7719 -7492 -7213 -6841 -6475 -6115 -5125 -5345 -4932 -4550	Moistu Degre Strain % 0.320 0.797 1.387 2.170 2.940 3.700 4.630 5.320 6.180 7.000	16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/ 17.19 17.35 17.49	0 ntent satura 67 2 19m/cm 0 0.152 0.302 0.450 0.596 0.739 0.880 1.018 1.152 1.257 1.418	Load		Strain	%	rea	57.	Load	Dial Rdg.	Strain	Area	67
Te: Load on Pan Oams 50 150 250 350 400 450 550	Dial Rdg .7871 .7719 .7492 .7213 .6841 .6475 .6115 .5725 .4932 .4550	Moistu Degre Strain % 0.320 0.797 1.381 2.170 2.940 3.700 4.630 5.320 6.180 7.000 7.750	16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/ 17.19 17.3/5 17.49 17.63 17.80	0 ntent satura Kgm/cm <sup>2</sup> 0,152 0,302 0,596 0,596 0,139 0,880 1,017 1,152 1,418 1,544	Load		Strain	%	rea	5.	Load	Dial Rdg.	Strain	Area	57
Te: Load on Pan  0 9ms 50 100 250 350 400 450 550 600	Dial Rdg -7871 -7719 -7492 -7213 -6841 -6475 -6115 -5125 -5345 -4932 -4191 -3791	Moistu Degre Strain % 0.320 0.797 1.387 2.170 4.630 5.3700 4.630 5.3700 6.180 7.750 8.610	16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/ 17.19 17.35 17.49 17.63 17.80 17.97	0 ntent satura 67 kgm/cm 0 0.152 0.302 0.450 0.596 0.739 0.880 1.018 1.152 1.287 1.418 1.544 1.670	Load		Strain	%	rea	51	Load	Dial Rdg.	Strain	Area	67
Te: Load on Pan  Ooms 50 100 250 350 450 450 550 650 700	Dial Rdg -7871 -7719 -7492 -7213 -6841 -6475 -6115 -5125 -5345 -4932 -4932 -4191 -3791 -3341 -2878	Moistu Degre Strain % 0.320 0.797 1.387 2.170 4.630 5.370 6.180 7.750 8.610 9.550 10.52	16.4 16.4 16.4 16.4 16.55 16.63 16.78 17.05/ 17.19 17.05/ 17.19 17.63 17.63 17.63 17.63 17.63 17.63 17.80 17.97 18.15 18.35	0.152 0.302 0.302 0.302 0.596 0.596 0.596 1.019 1.152 1.287 1.418 1.544 1.670 1.190 1.909	Load		Strain	%	rea	5.	Load	Dial Rdg·	Strain	Area	57
Te: Load on Pan  Doms 50 150 250 350 450 450 550 650 700 750	Dial Rdg .7871 .7719 .7492 .7213 .6841 .6475 .6115 .5125 .5346 .4932 .4550 .4191 .3791 .3341 .2878	Moistu Degre Strain % 0.320 0.797 1.381 2.170 4.630 5.370 4.630 7.750 8.610 9.550 10.52 11.69	16.4 16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/ 17.19 17.3/5 17.49 17.80 17.80 17.80 17.80 17.80 17.80 17.80 17.80 17.80 17.80 18.55	0.152 0.302 0.596 0.596 0.596 0.596 1.017 1.152 1.287 1.418 1.544 1.670 1.790 1.909 2.02	Load		Strain	%	rea	51	Load	Dial Rdg.	Strain	Area	57
Te: Load on Pan  Oams 50 150 250 350 450 450 550 600 650 700 750 800	Dial Rdg .7871 .7719 .7492 .7213 .6841 .6475 .6115 .5725 .4932 .4550 .4191 .3341 .2878 .2321 .1640	Moistu Degre Strain % 0.320 0.797 1.381 2.170 2.940 3.700 4.630 5.320 6.180 7.750 8.610 9.550 10.52 11.69 13.12	16.4 16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/ 17.19 17.35 17.49 17.63 17.63 17.63 17.80 17.80 17.80 17.80 17.80 17.80 17.80 18.80 18.80	0 ntent satura Kgm/cm 0 .152 0.302 0.596 0.596 0.596 1.018 1.152 1.418 1.418 1.544 1.670 1.190 1.909 2.02 2.12	Load		Strain	%	rea	51	Load	Dial Rdg.		Ared	67
Te: Load on Pan  Oams 50 100 250 350 400 450 550 600 650 700 750 800 850	Dial Rdg .7871 .7719 .7492 .7213 .6841 .6475 .6115 .5125 .5345 .4932 .4550 .4191 .3791 .3341 .2878 .2321 .1640 .1435	Moistu Degre Strain % 0.320 0.797 1.387 2.170 2.940 3.700 4.630 5.320 6.180 7.750 8.610 9.550 10.52 11.69 13.12 13.53	16.4 16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/ 17.19 17.35 17.49 17.80 17.80 17.97 18.15 18.35 18.59 18.97	0 ntent satura 67 2 19m/cm 0 0.152 0.302 0.450 0.596 0.596 0.739 0.880 1.018 1.152 1.257 1.418 1.544 1.670 1.190 1.909 2.02 2.12 2.24	Load		Strain	%	rea	51	Load	Dial Rdg.		Area	57
Te: Load on Pan  Oams 50 150 250 350 450 450 550 600 650 700 750 800 850	Dial Rdg .7871 .7719 .7492 .7213 .6841 .6475 .6115 .5725 .4932 .4550 .4932 .4550 .4191 .3341 .2878 .2321 .1640 .1435	Moistu Degre Strain % 0.320 0.797 1.381 2.170 2.940 3.700 4.630 5.320 6.180 7.750 8.610 9.550 10.52 11.69 13.12 13.53 15.2	16.4 16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/ 17.19 17.35 17.49 17.80 17.80 17.97 18.15 18.35 18.59 18.97	0 ntent satura 67 2 19m/cm 0 0.152 0.302 0.450 0.596 0.596 0.739 0.880 1.018 1.152 1.257 1.418 1.544 1.670 1.190 1.909 2.02 2.12 2.24	Load		Strain	%	rea	51	Load	Dial Rdg.		Area	67
Te: Load on Pan  Oams 50 150 250 350 450 450 550 600 650 700 750 800 850	Dial Rdg .7871 .7719 .7492 .7213 .6841 .6475 .6115 .5125 .5345 .4932 .4550 .4191 .3791 .3341 .2878 .2321 .1640 .1435	Moistu Degre Strain % 0.320 0.797 1.381 2.170 2.940 3.700 4.630 5.320 6.180 7.750 8.610 9.550 10.52 11.69 13.12 13.53 15.2	16.4 16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/ 17.19 17.35 17.49 17.80 17.80 17.97 18.15 18.35 18.59 18.97	0 ntent satura 67 2 19m/cm 0 0.152 0.302 0.450 0.596 0.596 0.739 0.880 1.018 1.152 1.257 1.418 1.544 1.670 1.190 1.909 2.02 2.12 2.24	Load		Strain	%	rea	51	Load	Dial Rdg.		Ared	57
Te: Load on Pan  Oams 50 150 250 350 450 450 550 600 650 700 750 800 850	Dial Rdg .7871 .7719 .7492 .7213 .6841 .6475 .6115 .5725 .4932 .4550 .4932 .4550 .4191 .3341 .2878 .2321 .1640 .1435	Moistu Degre Strain % 0.320 0.797 1.381 2.170 2.940 3.700 4.630 5.320 6.180 7.750 8.610 9.550 10.52 11.69 13.12 13.53 15.2	16.4 16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/ 17.19 17.35 17.49 17.80 17.80 17.97 18.15 18.35 18.59 18.97	0 ntent satura 67 2 19m/cm 0 0.152 0.302 0.450 0.596 0.596 0.739 0.880 1.018 1.152 1.257 1.418 1.544 1.670 1.190 1.909 2.02 2.12 2.24	Load		Strain	%	rea	51	Load	Dial Rdg.		Ared	67
Te: Load on Pan  Oams 50 150 250 350 450 450 550 600 650 700 750 800 850	Dial Rdg .7871 .7719 .7492 .7213 .6841 .6475 .6115 .5725 .4932 .4550 .4932 .4550 .4191 .3341 .2878 .2321 .1640 .1435	Moistu Degre Strain % 0.320 0.797 1.381 2.170 2.940 3.700 4.630 5.320 6.180 7.750 8.610 9.550 10.52 11.69 13.12 13.53 15.2	16.4 16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/ 17.19 17.35 17.49 17.80 17.80 17.97 18.15 18.35 18.59 18.97	0 ntent satura 67 2 19m/cm 0 0.152 0.302 0.450 0.596 0.596 0.739 0.880 1.018 1.152 1.257 1.418 1.544 1.670 1.190 1.909 2.02 2.12 2.24	Load		Strain	%	rea	51	Load	Dial Rdg.		Ared	67
Te: Load on Pan  Oams 50 150 250 350 450 450 550 600 650 700 750 800 850	Dial Rdg .7871 .7719 .7492 .7213 .6841 .6475 .6115 .5725 .4932 .4550 .4932 .4550 .4191 .3341 .2878 .2321 .1640 .1435	Moistu Degre Strain % 0.320 0.797 1.381 2.170 2.940 3.700 4.630 5.320 6.180 7.750 8.610 9.550 10.52 11.69 13.12 13.53 15.2	16.4 16.4 16.4 16.4 16.55 16.63 16.78 16.91 17.05/ 17.19 17.35 17.49 17.80 17.80 17.97 18.15 18.35 18.59 18.97	0 ntent satura 67 2 19m/cm 0 0.152 0.302 0.450 0.596 0.596 0.739 0.880 1.018 1.152 1.257 1.418 1.544 1.670 1.190 1.909 2.02 2.12 2.24	Load		Strain	%	rea	51	Load	Dial Rdg.		Ared	67



	UNIVE DEP TO SOIL TRI-A	of (	CIVIL	ENG	BORAT	RING		SITE SAMPL LOCAT HOLE	ION #		£ 141	DEPTH	30'	
Mach	ine Da	ta:-						Marie Contract		on of S		-	70 7	
Mach	ine No.		5					GLA	CIA	L TILL	SI	ty. 10.	n to	
Multi	plicatio	n Fac	tor_	XIC	0		1.0			plastic				
Wt. Lo	ading	Block	+ Pist	ton (gm	ıs.)			COA	1 &	pea	gray	iel pr	esey	it.
					SPECII	MEN			DAT		J			
Spec	cimen N	lumbe	r	-	01 2,011			2	-	3	4	5	1	6
	eral P			67% )								-		0
Len	-			nches			4.6							
Are	0 D=	1.8"		The state of the s			16.4							
	me			C. C. S.			191.5							,
Dry	Unit We	eight			†.		131.0							
G <sub>S</sub> =	· Vc	lume	Soil	Solids										
	Tare +												12	
	Tare +						492.63	5						
Wt.	Tare +	Soil					432.8							
Num	ber an	d wei	ght of	Tare			30.7	٥						
	Soil						402.1							
Befo	ra	Weigh	t of	water										
		Moist	ire c	ontent										
Tes				saturo										
Aft	er	Weigh	t of	water			59.8	3						
Tes		Moistu	ire c	ontent			6.7 %							
16:		Degre	e of	satura	ition									
Load	Dial				Load	Dial	T	T		Load	Dial			
on Pan	Rdg	Strain	Area	07	on Pan	Rdg	Strain	Area	5.	on Pan	Rdg	Strain	Area	51
1 011		/8	CIK	Kgm/cm	1 011	3					3			
0	.8870		16.4											
50	·8600													
100	.8215					· · · · · · · · · · · · · · · · · · ·				-				
150	.788Z .753Z												-	
250	.7031													
300	.6548	1												
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LOGBOITS MINERSITY OF ALBERTA THE RESERVE OF THE PARTY OF THE DEPTH OF CIVIL ENGINEERING DIBWAS 100174001 SCIL MESHAVICS LABORATORY 3100 107530 TRIAMBLE COMPRESSION MALTIN KOET DATE Trung to autains. -nine entrae ochine No\_\_\_\_ 1000 ultiplication Factor A THE STATE OF THE PARTY OF THE 4 t. Loading Block + Piston (gms). ATAG SPECIMEN 3% 1 Specimen Number Gieral Pressure (dos) inches ength. sq. cms. 081/ Fig. UP BIOS Aggrega C. C. S. enulch by Unit Weight Ibs/cuiff. 's= Volume Soil Solids VI. Tor + Soil + Warny of story V Tor. + Sall + Water a end 40.00 liot toT.1" Junto r and wright of Torn otae. Mt. Soil Weight of woter 3 101 8 Molsture content 1251 Degree of saturation Weight of weter TOTTA Moisturn contant X 110 tauT Organa of soluration 0001 00 7,000 40 IDIO DICI Dig TO TO YOUTA MINITE Mena Informa Strein, Aron n es 0.017 10 Rdg ROO nog - <= .... 175 . . 5 THE R. LEWIS CO. - 1. mc C 84 20 05 La lucal III agia -5-41135

UNIVERSITY DEP'T of CIVI SOIL MECHANIC	L ENGIN	ERTA EERING RATORY	SITE SAMPLE LOCATION		F 144+	STREET.
MOISTURE	CON.	TENT	TECHNI	CIAN P.K	DEPT DATE	
Hole No.	3	3	3	3	3	3
Depth	21/2	5'	11/2	0	121/2	15
Sample No-		1		2		3
Container No-	1A27	1417	1A53	1A17	1217	1A27
Wt. Sample Wet + Tare	68.76	15.30	85.65	81.08	109.11	65.83
Wt·Sample Dry + Tare	55.36	61.10	67.52	64.05	84:32	52.61
Wt· Water	13.40	14.20	18.13	17.03	24.79	13.22
Tare Container	17.69	17.43	17.56	17.43	17.43	17.66
Wt. of Dry Soil	37.67	43.67	49.96	46.62	66.89	34.95
Moisture Content w %	35.6	37.6	36.4	39.6	37.0	38.8
Hole No	3	3	3	3	3	3
Depth	171/2	20	221/2	25'	26	27'
Sample No-		4		5		
Container No-	1A23	1A27	1A35	1AZ7	1A30	78
Wt Sample Wet + Tare	104.88	85.59	102.33	75.57	115.34	100.92
Wt Sample Dry + Tare	80.19	70.15	79.06	66.91	101.01	90.55
Wt· Water	24.69	19.44	23.27	8.66	14. 33	10.37
Tare Container	17.27	17.66	17.71	17.66	17.56	24.52
Wt. of Dry Soil	62.92	52.49	61.35	49.25	83.45	66.03
Moisture Content w%	39.3	37.1	38.0	17.6	17.Z	15.1
Hole No.	3					
Depth	30					
Sample No	6					
Container No-	7C					
Wt Sample Wet + Tare	101.89					
Wt-Sample Dry + Tare	92.40	-				
Wt· Water	9.49					
Tare Container	24.52					
Wt. of Dry Soil	67.88					
Moisture Content 6%	14.0					

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## LABORATORY TEST RESULTS

TEST PILE NO. 4



UNIVERSITY of AL	BERTA	PROJEC	TTEST PI			
DEP'T of CIVIL ENGI	VEERING	SAMPLE	4th AVE	E 144	th STR	EET.
	RATORY	LOCATIO				
	IMITS	HOLE	# 4	1	DEPTH	3'
ATTENDENO L	TIVIL 1 3	TECHNI	CIAN P.+		DATE /	
Li	quid Limit					
Trial No.	2	3	1	Z		3
No. of Blows 41	39	42	18	18		7
Container No. V71	V46	V36	V 41	V79		120
Wt. Sample Wet + Tare 86.7321	97.9521	84.0857	84.0935	94.31	40 81	.1706
Wt. Sample Dry + Tare 80.0564	91.4993	76.3605	77.2037	87.27	07 73	5.7135
Wt. Water 6.6757	6.4528	7.7252	6.8898	7.04	433 7	1.4571
Tare Container 72.0433	83.7237	67.0527	69.6448	79.50	047 6	5.5906
Wt. of Dry Soil 8.0131	7.7756	9.3078	7.5589	7.76	60 8	3.1229
Moisture Content w% 83.4	83.1	83.0	91.Z	90.	7	91.7
	rage Values		Plastic I	_imit		
	rage Values	Trial No.			2	3
	7= B7.9 %	Container	No.	4	7	11
n and a second	p= 29.7 %	Wt. Sample		34.0925	32.6441	42.2254
	•	Wt. Sample	Dry + Tare	33.1327	32.0180	41.5078
	•	Wt. Water		0.9598	0.6261	0.7176
	58.2 %	Tare Cont	ainer	29.8903	29.9014	39.1062
91	* 22.7	Wt. of Dry				2.4016
I	= 2.56	Moisture C	ontent %	Z9.6	29.7	29.9
			Shrinkage	Limit		
90		Trial No.				
		Container				
89		Wt. Sample				
		Wt Sample	Dry + lare			
8		Wt. Water				-
88		Tare Cont	,			
-		Wt. of Dry				
v <sub>81</sub>		Moisture Co Vol. Contai				
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UNIVERSITY OF ALBERTA
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UNIVERSITY of	ALB	ERTA	PROJECT	T TEST	PILE #	4 -		
DEP'T of CIVIL E				#Z	€ 144	1 27	REET.	
SOIL MECHANICS			SAMPLE					
			HOLE			DEPTH	8'	
ATTERBERG	LI	MITS	TECHNI				1/29/59	
	Liq	uid Limit						
Trial No.		2	3	1	2		3	
No. of Blows 44		4-3	42	16	17		16	
Container No.		Z	A4	4 AIZ		>	V70	
Wt. Sample Wet + Tare 88.6	723	91.6480	94.5925	88.6320	98.4	865	97.5475	
Wt. Sample Dry + Tare 82.7	741	84.9544	87.3712	81.Z685	5 91.Z	905 9	70.9651	
	982	التناف فيستنف فيتناف المناف ال	7.2213	7.3635			6.58z4	
	352		76.5604				8z./483	
	389	9.9918	10.8108			088	8.8168	
Moisture Content w% 67.	4	67.0	66.8	74.4	73.	8	74.7	
	Aver	age Values		Plastic	Limit			
	11	7	Trial No.		1	2	3	
		= 71.1 %	Container		1	2	3	
	Tw.		Wt Sample					
	Ws	=	Wt. Sample	Dry + Tare		•		
	H	1.09	Wt. Water				0 0.8760	
	<sup>1</sup> p		Tare Cont				9143.8264	
15	If		Wt. of Dry Soil 3.4871 4.0264 2					
	I+	= 2.39	Moisture Content % 29.4 29.6 29.9					
	# .			Shrinkage	Limit	1		
74			Trial No.					
			Container					
13			Wt Sample					
			Wt Sample	Dry + lare				
2			Wt. Water					
8° 72			Tare Cont			-		
			Wt. of Dry					
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UNIVERSITY	of ALE	BERTA	PROJEC	I TEST	PILE #	4	
			SITE	14th AVE	£ 144+	h STRE	ET.
DEP'T of CIVI	L ENGIN	EERING	SAMPLE	#3			
SOIL MECHANIC	S LABO	RATORY	LOCATI	ON			
ATTERBER	G LI	MITS	HOLE	#4			13'
			TECHNI	CIAN P.	K	DATE /	30/59
	Liq	uid Limit					
Trial No.	1	2	3				
No. of Blows	24	24	26				
Container No.	V20	V71	¥79				
Wt. Sample Wet + Tare	81.3943	87.1538	96.3688				
Wt. Sample Dry + Tare	75.1911	81.2461	89.1317				
Wt. Water	6.2032	5.9077	6.6371				
Tare Container	65.5906	72.0433	19.5047				
Wt of Dry Soil	9.6005		10.2270		1		
Moisture Content w%	64.6	64.2	64.8				
				Diactic	1::4		
	7 4 5 7 8 7 8	age Values	7	Plastic		2	7
	w.	= 64.5 %	Trial No.	<b>A1</b> -	- /	4	3
		3013/	Confainer	NO.	4	2/1/20	11
		= 30.1 %					
	w <sub>s</sub>		Wt. Sample	Dry + lare			
	7_	= 34.4%	Wt. Water	•			1.2020
					29.8903		
	4	*	Wt of Dry			3.2746	
	I <sub>†</sub>	2	Moisture C		30.4		29.8
				Shrinkage	Limit		
			Trial No.				
			Container	No.			
			Wt.Sample	Wet + Tare			
			Wt Sample I	Dry + Tare			
%			Wt-Water				
			Tare Cont	ainer			
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No. of Blows  32  31  29  15  16  16  16  Container No.  W36  V71  V46  V41  V79  V20  Wt. Sample Wet + Tare  21.767  29.1555  10.9069  85.1794  98.7345  82.5908  Wt. Sample Dry + Tare  71.0070  83.6811  95.6194  79.6592  97.8805  76.5677  Wt. Water  5.1691  6.0744  6.2275  5.5202  6.8538  6.023  Tare Container  61.0527  72.0433  83.7231  69.6448  79.5047  65.5908  Wt. of Dry Soil  9.9543  11.6378  11.9557  10.0144  72.3758  10.9777  Moisture Content w%  51.9  52.3  52.0  55.2  55.4  55.0  Wt. Sample Wet + Tare  31.123  19.6404  Wt. Sample Wet + Tare  32.9763  32.4760  Wt. Sample Dry + Tare  33.1237  Wt. Sample Dry + Tare  Wt. Sample Dry + Tare  Wt. Sample Dry + Tare  Wt. Sample Wet + Tare  Wt. Sample Dry + Tare	UNIVERSITY	of ALE	BERTA	SITE	LT DVE	LE 4	S~:	200		
SOIL MECHANICS LABORATORY	DEP'T of CIV	L ENGIN	EERING			5 144	- 3.LK	SEEL.		
## HOLE ## TECHNICIAN P.K. DATE 2/1/59    Trial No.					-					
Trial No.	ATTERRED	G LI	MITC		#4		DEPT	H 181		
Liquid Limit   Z   3	ATTENDER	G LI	IVII I D	TECHNI	CIAN P.			-		
No. of Blows  32 31 29 15 16 16  Container No.		Liq	uid Limit							
Container No.	Trial No.	1	Z	3	1	. 2		3		
Wit Sample Wet + Tare       QX.176.7       QA.1555       IO.196.9       QB.1794       QB.1794 <th cols<="" td=""><td>No. of Blows</td><td>32</td><td>31</td><td colspan="2">29 15</td><td colspan="2">16</td><td colspan="2">16</td></th>	<td>No. of Blows</td> <td>32</td> <td>31</td> <td colspan="2">29 15</td> <td colspan="2">16</td> <td colspan="2">16</td>	No. of Blows	32	31	29 15		16		16	
Wt. Sample Dry + Tore       77.0070       83.6811       95.6794       79.6592       97.8805       76.567         Wt. Water       5.1671       6.0794       6.2275       5.5202       6.8538       6.023         Tare Container       51.0527       72.0453       83.7237       69.6448       79.5047       65.500         Wt. of Dry Soil       9.9643       11.6378       11.9557       10.0144       72.3758       10.977         Moisture Content w%       5.9       52.3       52.0       55.2       55.4       55.0         Wt. Sample Content w%       5.9       52.3       52.0       55.2       55.4       55.0         So.       10	Container No.	V36	V71	V46	V70	7				
## Wit Water	Wt· Sample Wet + Tare	82.1767	89.1555	101.9069				82.5908		
With water	Wt. Sample Dry + Tare	77.0070	83.6811	95.6794						
Tare   Container   C7.0527   72.0433   83.7237   69.6448   79.5047   65.5901   9.9511   1.6378   11.6378   11.9557   10.0144   72.3758   10.9777   65.5901   7.9777   65.5901   7.9777   65.5901   7.9777   65.5901   7.9777   7.00144   72.3758   7.9777   7.00144   72.3758   7.9777   7.00144   72.3758   7.9777   7.00144   72.3758   7.9777   7.00144   72.3758   7.9777   7.00144   72.3758   7.9777   7.00144   72.3758   7.9777   7.00144   72.3758   7.9777   7.00144   7.23758   7.9777   7.0014   7.001	Wt· Water									
With of Dry Soil   9.9543   11.6378   11.9557   10.0144   12.3758   10.9771	Tare Container									
Moisture Content w%   S/9   S2.3   S2.0   S5.2   S5.4   S5.0										
Average Values	Moisture Content w%						-			
				1					===	
W = 53.0 %   W = 24.7 %   W + Sample WeitTare 33.74.39 33.1.2.1 W. 89				<b>T</b>		LIMIT	2	2		
Wp = 24.7 %   Wt. Sample Wet+Tare   33.74.37   33.12.21   W. 89t   Ws =		14	= 53.0%	Costs:	NI o	./				
Wish   Sample Dry + Tare   32.9763   32.4760   41.34     Ip = 28.3 %     Ip = 28.3 %     Ip = 11.2     Ip = 2.52     Moisture Container   No.     Wish Sample Wet + Tare     Wish Sample Dry + Tare     Wish Sam			21179	Container	Weal Ton	22 7/120		21 111	001/2	
To		w w	p= 24.1 10	Wit Sample	Wettidre	33.7439	33.12	21 44.	3943	
Ip = 28.3 %   Wi. Water   Tare   Container   29.8903 29.90 1/39.106   3.50cm   2.5746   2.23   Moisture Content %   24.8   24.7   24.6   Shrinkage Limit   Trial No.   Container No.   Wi. Sample Wet + Tare   Wi. Sample Dry + Tare   Wi. Water   Wi. Water   Tare   Wi. Worder   Tare   Wi. Worder   Tare   Wi. Worder   Tare   Wi. Worder   Tare   Container   Wi. Of Dry Soil W.   Moisture Content \( \psi \) \(	56	W <sub>S</sub>	=		Dry + lare					
					•					
Solution   Solution										
Shrinkage Limit  Trial No.  Container No.  Wt. Sample Wet + Tare  Wt. Water  Tare Container  Wt. of Dry Soil Wo.  Moisture Content w.%  Vol. Container  Vol. Dry Soil Pat Vo.  Shrinkage Vol. V-Vo.  Shrinkage Vol. V-Vo.  Shrinkage Limit ws  Limit  Trial No.  Container No.  Wt. Sample Wet + Tare  Wt. Water  Tare Container  Vol. Container  Vol. Container  Vol. Dry Soil Pat Vo.  Shrinkage Vol. V-Vo.  Shrinkage Limit ws  Limit  We Shrinkage Limit ws  Limit  Woll Container  Vol. Dry Soil Pat Vo.  Shrinkage Limit ws  Limit  We Shrinkage Limit ws  Remarks:  Remarks:  Remarks:		If If					1			
Shrinkage Limit  Trial No.  Container No.  Wt. Sample Wet + Tare  Wt. Water  Tare Container  Wt. of Dry Soil Wo.  Moisture Content w%.  Vol. Container V  Vol. Dry Soil Pat Vo.  Shrinkage Vol. V-Vo.  Shrinkage Limit Ws.   2 = w (V-Vo. x 100)  Description of Sample:  CLAY: highly plastic stiff  moist, grey in color.		I,	= 2.52							
Container No.  Wt. Sample Wet + Tare  Wt. Water  Tare Container  Wt. of Dry Soil Wo  Moisture Content w%  Vol. Container V  Vol. Dry Soil Pat Vo  Shrinkage Vol. V-Vo  Shrinkage Limit Ws   Description of Sample:  CLAY: highly plastic, stiff  moist, grey in color.					<u>Shrinkage</u>	Limit				
Wt. Sample Wet + Tare  Wt. Water  Tare Container  Wt. of Dry Soil Wo  Moisture Content w%  Vol. Container V  Vol. Dry Soil Pat Vo  Shrinkage Vol. V-Vo  Shrinkage Limit Ws	55	9\		Trial No.						
Wt. Sample Dry + Tare  Wt. Water  Tare Container  Wt. of Dry Soil Wo  Moisture Content w%  Vol. Container V  Vol. Dry Soil Pat Vo  Shrinkage Vol. V-Vo  Shrinkage Limit Ws   Ws = w (V-Vo x 100)  Description of Sample:  CLAY: highly plastic, stiff  moist, grey in color.				Container	No.					
Wt. Water Tare Container Wt. of Dry Soil Wo Moisture Content w% Vol. Container Vol. Dry Soil Pat Vo Shrinkage Vol. V-Vo Shrinkage Limit Ws  Ws = w (V-Vo x 100)  Description of Sample: CLAY: highly plastic stiff moist, grey in color.  Remarks:										
Tare Container  Wt. of Dry Soil Wo  Moisture Content w%  Vol. Container V  Vol. Dry Soil Pat Vo  Shrinkage Vol. V-Vo  Shrinkage Limit Ws   Description of Sample:  CLAY: highly plastic, stiff  moist, grey in color.		<del></del>		Wt Sample	Dry + Tare					
Moisture Content W%  Vol. Container V  Vol. Dry Soil Pat Vo  Shrinkage Vol. V-Vo  Shrinkage Limit Ws   Ws = w (V-Vo x 100)  Description of Sample:  CLAY: highly plastic, stiff  moist, grey in color.  Remarks:				Wt Water				_		
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PROJECT 113 PAMPLE LOCATION 310H

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DEP'T of CIVI		FERING		4th AVE &	144,	n S-	TREET.	
SOIL MECHANIC			SAMPLE					
			HOLE	#4		DEPT	H 25'	
ATTERBER	G LI	MITS	TECHNI	CIAN P.			1/8/59	
	Liq	uid Limit						
Trial No.	1	2	3	1	2		3	
No. of Blows	44	44	45 21		21		19	
Container No.	V50	V70	1	A4		AIZ		
Wt· Sample Wet + Tare	98.9810	97.9526	91.0912	91.7674	92.64	81	88.0427	
Wt. Sample Dry + Tare	93.4766	92.9262	85.6984	86.1965	87.32	64	82.4810	
Wt. Water	5.5104	5.0264	5.3928	5.5709	5.32	17	5.5617	
Tare Container	81.5525	82.1483	74.0352	74.9626	76.56	04	71.3514	
Wt of Dry Soil	11.9241	10.7779	11.6632	11.2339	10.76			
Moisture Content w%	46.1	46.6	46.Z	49.6	4-9.	4	49.9	
		ana Valuos		Plastic I	_imit			
		age Values	Trial No.		1	2	3	
	w	= 48.7 %	Container	No.	1	Z	3	
	w	20.3 %	Wt. Sample	Wet+Tare	51.0806	51.48	186 49.3948	
			Wt. Sample	Dry + Tare	50.4181	50.68	32548.4593	
	w <sub>s</sub>	-	144 141-4				061 0.9355	
	Ip	= 28.4%	Tare Cont	ainer	47.1130	46.78	891 43.8Z64	
50	14	* 10.Z	Wt of Dry	Soil	3.3051	3.89	34 4.6329	
30		= 2.8	Moisture Content % 20.0 20.7 20.2					
	1		Shrinkage Limit					
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			Shrinkage	Limit We				
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SOIL MECHANICS	LABOR	RATORY	SAMPLE				
ATTERBERG	LABOI	MITS	HOLE	<b>#</b> 4		DEPTH	73
ATTENDENO	<u> </u>	IVIII 1 3	TECHNI				1/10/59
	Liq	uid Limit				,	[ ] [ ]
Trial No.	/	2	3	,	2		2
No. of Blows	39	39	37	15			3
Container No.	AIZ	V70	A4	17		17	
1111 0		106.8438	101.8405	96.0438	99.36		7.1658
	8.6530	98.6050	93.4759	88.3855			4.9571
1424 142 4	8.635Z	8.z388	8.3646	1.6583			7.2087
Tare Container 7/	1.3514	82.1483	76.5604				1.5525
	7.3026	16.4567	16.9155	14:3503	15.38		3.4046
Moisture Content w%	49.8	50.1	49.4	53.4	53.0		53.6
		an Value		Plastic L	imit		
	<del>- - - - </del>	ige Values	Trial No.		/	Z	3
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	ws:		Wt. Sample				
			Wt. Water				1.0425
	Îp :	26.7%	Tare Cont	ainer 4	7.1130	46.7891	43.8264
	I4 2	10.4	Wt of Dry Soil 4.8844 4.889				4.1059
		2.57	Moisture Co	ontent %	24.8	25.0	25.3
	<del>                                      </del>		·	Shrinkage	Limit		
			Trial No.				
			Container I	No.			
54			Wt Sample V				
			Wt Sample D	ry + Tare			
8			Wt Water				
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=			Wt. of Dry S			-	
υ 5z			Moisture Co				-
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			Remarks:				
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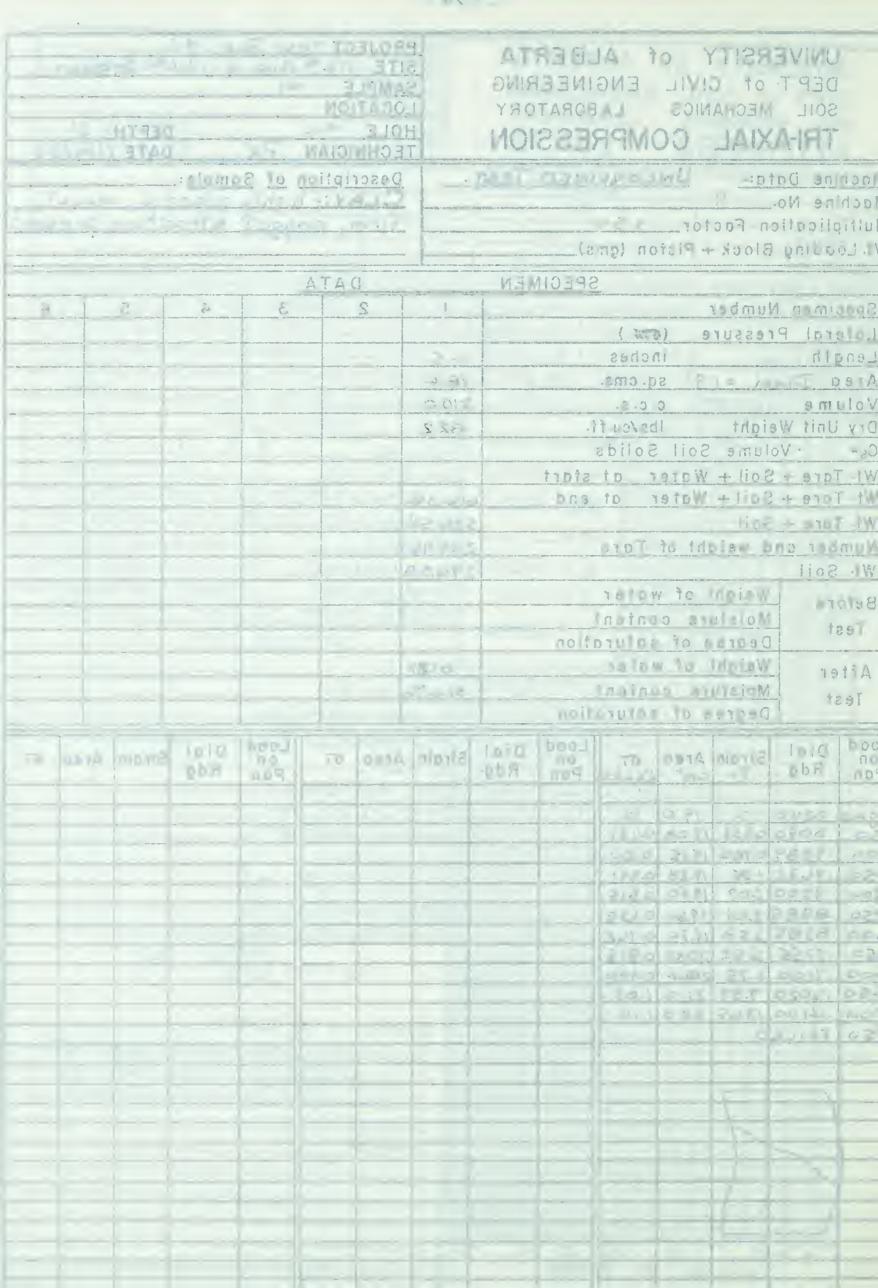
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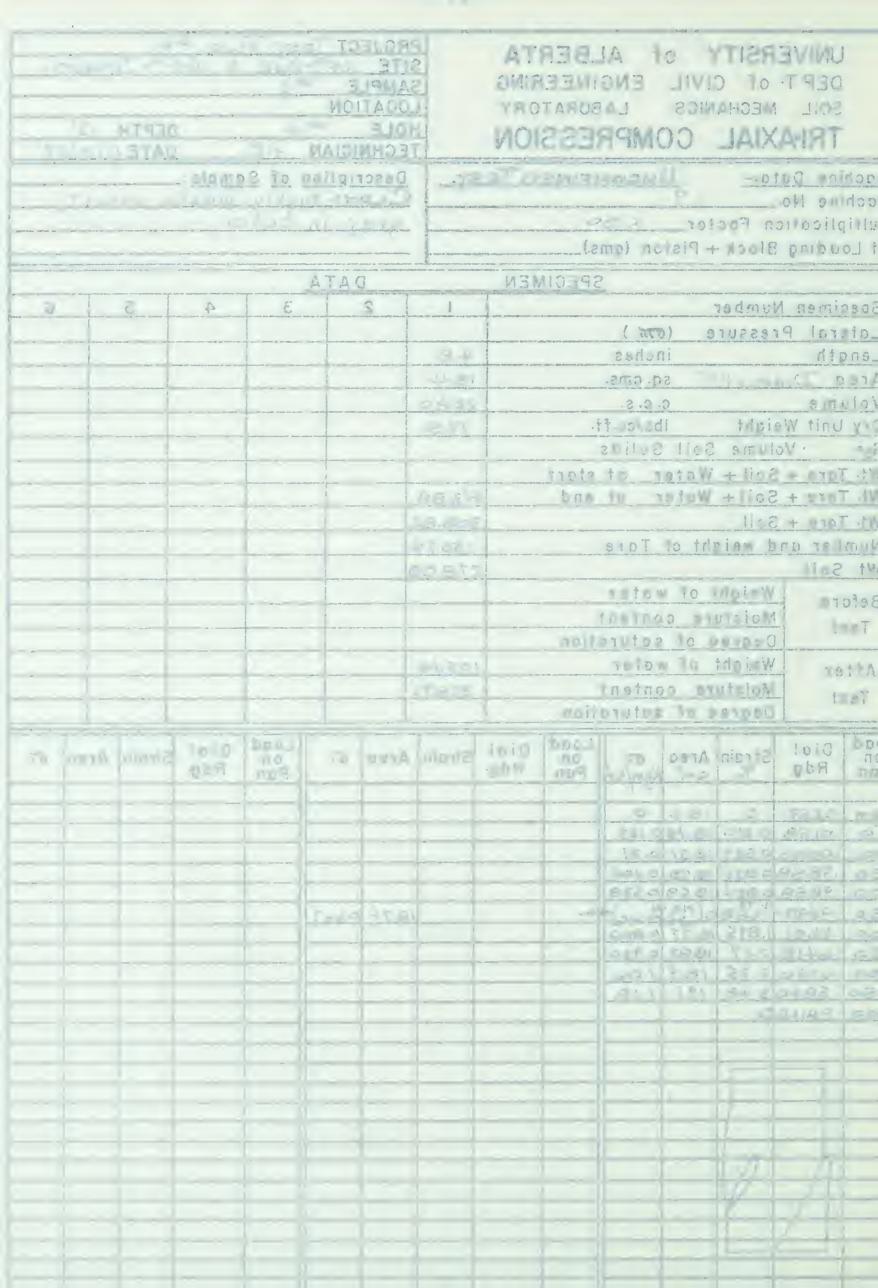
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Tes	st	Moistu Degre	e of	ontent satura	ition	Dial	31.6	%			Load	Dial			T	
Tes Load	Dial	Moistu Degre Strain	e of	ontent satura	Load	Dial	31.6	%	Area	5.	Load	Dial Rda	Strain	Are	a 51	
Tes	st	Moistu Degre Strain	e of	ontent satura	Load	Dial Rdg	31.6	%	Area	5.	Load on Pan	Dial Rdg.	Strain	Are	0 57	
Load on Pan	Dial Rdg	Moistu Degre Strain	e of Area	ontent satura or Kam/cm	Load		31.6	%	Area	5.	on		Strain	Are	a 67	
Load on Pan	Dial Rdg	Moistu Degre Strain	Area cm²	ontent satura or Kam/cm	Load		31.6	%	Area	5.	on		Strain	Are	a 57	
Load on Pan	Dial Rdg	Moistu Degre Strain	Area cm² 19.0	ontent satura or Kam/cm	Load		31.6	%	Area	57.	on		Strain	Are	a 57	
Load on Pan Ogms 50	Dial Rdg .0240 .0090 .9889	Strain  O  O.3333  O.780	19.08 19.15	ontent satura or Kam/cm 0.131 0.261 0.391	Load		31.6	%	Area	51	on		Strain	Are	a 67	
Load on Pan Ogms 50 100 150 Zoo	Dial Rdg .0240 .0090 .9889 .9632	Moistu Degre Strain % 0.333 0.780 1.35 2.09	19.0 19.08 19.15 19.40	07 Kqm/cm 0.131 0.261 0.391 0.515	Load		31.6	%	Area	571	on		Strain	Are	a 57	
Load on Pan Ogms 50 100 250	Dial Rdg .0240 .0090 .9889 .9632 .9300	Strain 9. 0.333 0.780 1.35 2.09 3.01	19.0 19.08 19.15 19.40 19.60	07 Kqm/cm 0.131 0.261 0.391 0.515 0.638	Load		31.6	%	Area	57.	on		Strain	Are	0 67	
Load on Pan Ogms 50 100 150 250 300	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885	Moistu Degre Strain % 0.333 0.780 1.35 2.09 3.01 3.58	19.08 19.18 19.40 19.60 19.10	0.131 0.261 0.515 0.762	Load		31.6	%	Area	51	on		Strain	Are	a 67	
Tes Load on Pan Ogms 50 100 150 250 350	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885 .8385	Moistu Degre Strain 0.333 0.780 1.35 2.09 3.58 5.52	19.0 19.08 19.15 19.40 19.40 19.40 19.40 20.01	0.131 0.261 0.515 0.638 0.762	Load		31.6	%	Area	57.	on		Strain	Are	a 57	
Tes Load on Pan Ogms 50 100 150 250 300 350 400	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885 .1755	Moistu Degre Strain % 0 0.333 0.780 1.35 2.09 3.01 3.58 5.52 6.75	19.08 19.08 19.15 19.40 19.60 19.60 19.70 20.01 20.4	0.131 0.261 0.515 0.815 0.980	Load		31.6	%	Area	51	on		Strain	Are	a 67	
Tes  Load on Pan  Ogms 50 100 250 250 350 400 450	Dial Rdg .0240 .0090 .9889 .9300 .8885 .7155 .7100 .6020	Moistu Degre Strain 9.0 0.333 0.780 1.35 2.09 3.58 5.52 6.75 9.39	19.0 19.0 19.08 19.15 19.28 19.40 19.60 19.10 20.01 20.4 21.0	0.131 0.261 0.515 0.638 0.762 0.875 0.980	Load		31.6	%	Area	57.	on		Strain	Are	a 57	
Tes  Load on Pan  Ogms 50 100 150 250 350 400 450 500	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885 .1755 .7100 .6020	Moistu Degre Strain % 0 0.333 0.780 1.35 2.09 3.01 3.58 5.52 6.75 9.39 13.65	19.0 19.0 19.08 19.15 19.28 19.40 19.60 19.10 20.01 20.4 21.0	0.131 0.261 0.515 0.638 0.762 0.875 0.980	Load		31.6	%	Area	57	on		Strain	Are	a 57	
Tes  Load on Pan  Ogms 50 100 150 250 350 400 450 500	Dial Rdg .0240 .0090 .9889 .9300 .8885 .7155 .7100 .6020	Moistu Degre Strain % 0 0.333 0.780 1.35 2.09 3.01 3.58 5.52 6.75 9.39 13.65	19.0 19.0 19.08 19.15 19.28 19.40 19.60 19.10 20.01 20.4 21.0	0.131 0.261 0.515 0.638 0.762 0.875 0.980	Load		31.6	%	Area	57.	on		Strain	Are	a 57	
Tes  Load on Pan  Ogms 50 100 150 250 350 400 450 500	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885 .1755 .7100 .6020	Moistu Degre Strain % 0 0.333 0.780 1.35 2.09 3.01 3.58 5.52 6.75 9.39 13.65	19.0 19.0 19.08 19.15 19.28 19.40 19.60 19.10 20.01 20.4 21.0	0.131 0.261 0.515 0.638 0.762 0.875 0.980	Load		31.6	%	Area	57.	on		Strain	Are	0 67	
Tes  Load on Pan  Ogms 50 100 150 250 350 400 450 500	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885 .1755 .7100 .6020	Moistu Degre Strain % 0 0.333 0.780 1.35 2.09 3.01 3.58 5.52 6.75 9.39 13.65	19.0 19.0 19.08 19.15 19.28 19.40 19.60 19.10 20.01 20.4 21.0	0.131 0.261 0.515 0.638 0.762 0.875 0.980	Load		31.6	%	Area	51	on		Strain	Are	a 57	
Tes  Load on Pan  Ogms 50 100 150 250 350 400 450 500	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885 .1755 .7100 .6020	Moistu Degre Strain % 0 0.333 0.780 1.35 2.09 3.01 3.58 5.52 6.75 9.39 13.65	19.0 19.0 19.08 19.15 19.28 19.40 19.60 19.10 20.01 20.4 21.0	0.131 0.261 0.515 0.638 0.762 0.875 0.980	Load		31.6	%	Area	57.	on		Strain	Are	0 67	
Tes  Load on Pan  Ogms 50 100 150 250 350 400 450 500	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885 .1755 .7100 .6020	Moistu Degre Strain % 0 0.333 0.780 1.35 2.09 3.01 3.58 5.52 6.75 9.39 13.65	19.0 19.0 19.08 19.15 19.28 19.40 19.60 19.10 20.01 20.4 21.0	0.131 0.261 0.515 0.638 0.762 0.875 0.980	Load		31.6	%	Area	57	on		Strain	Are	9 67	
Tes  Load on Pan  Ogms 50 100 150 250 350 400 450 500	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885 .1755 .7100 .6020	Moistu Degre Strain % 0 0.333 0.780 1.35 2.09 3.01 3.58 5.52 6.75 9.39 13.65	19.0 19.0 19.08 19.15 19.28 19.40 19.60 19.10 20.01 20.4 21.0	0.131 0.261 0.515 0.638 0.762 0.875 0.980	Load		31.6	%	Area	57	on		Strain	Are	a 57	
Tes  Load on Pan  Ogms 50 100 150 250 350 400 450 500	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885 .1755 .7100 .6020	Moistu Degre Strain % 0 0.333 0.780 1.35 2.09 3.01 3.58 5.52 6.75 9.39 13.65	19.0 19.0 19.08 19.15 19.28 19.40 19.60 19.10 20.01 20.4 21.0	0.131 0.261 0.515 0.638 0.762 0.875 0.980	Load		31.6	%	Area	57	on		Strain	Are	a 67	
Tes  Load on Pan  Ogms 50 100 150 250 350 400 450 500	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885 .1755 .7100 .6020	Moistu Degre Strain % 0 0.333 0.780 1.35 2.09 3.01 3.58 5.52 6.75 9.39 13.65	19.0 19.08 19.15 19.28 19.40 19.60 19.10 20.01 20.4 21.0	0.131 0.261 0.515 0.638 0.762 0.875 0.980	Load		31.6	%	Area	57	on		Strain	Are	0 57	
Tes  Load on Pan  Ogms 50 100 150 250 350 400 450 500	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885 .1755 .7100 .6020	Moistu Degre Strain % 0 0.333 0.780 1.35 2.09 3.01 3.58 5.52 6.75 9.39 13.65	19.0 19.08 19.15 19.28 19.40 19.60 19.10 20.01 20.4 21.0	0.131 0.261 0.515 0.638 0.762 0.875 0.980	Load		31.6	%	Area	57	on		Strain	Are	0 67	
Tes  Load on Pan  Ogms 50 100 150 250 350 400 450 550	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885 .7755 .7100 .6020 .4100 FAILE	Moistu Degre Strain % 0 0.333 0.780 1.35 2.09 3.01 3.58 5.52 6.75 9.39 13.65 D.	19.0 19.08 19.15 19.28 19.40 19.60 19.70 20.01 22.0	0.131 0.261 0.391 0.515 0.638 0.762 0.875 0.980 1.07	Load on Pan	Rdg	Strain	20			Pan	Rdg				
Tes  Load on Pan  Ogms 50 100 150 250 350 400 450 550	Dial Rdg .0240 .0090 .9889 .9632 .9300 .8885 .1755 .7100 .6020	Moistu Degre Strain % 0 0.333 0.780 1.35 2.09 3.01 3.58 5.52 6.75 9.39 13.65 D.	19.0 19.08 19.15 19.28 19.40 19.60 19.70 20.01 22.0	0.131 0.261 0.391 0.515 0.638 0.762 0.875 0.980 1.07	Load on Pan	Rdg	Strain	20			Pan	Rdg				



	UNIVE	RSIT	Y o	f Al	BER		SITE 114th ANE & 144th STREET.							
				ENG			SITE ) SAMPL	1412	HUE F1	£ 140	F'L 5	TREE	. 7	
				LA				LOCAT		~				
								HOLE		F		DEPTH	8'	
	INIA	NIAL		OMPR	(E22	NOI				P.K				159
Mach	ine Da	ta:-	U	NCON	FINE	DTE								
Mach	ine No		9					CLA	y h	ahly	lastic	- mo	st I	irm.
Multi	plication	n Fac	tor_	x 50				nugget structure present.						
Wt. Lo	ading	Block	+ Pist	on (gm	s.)			,	)			1		
						AFN								
Spec	im on I	lumba.		-	SPECII	WEN		_	ATA	_				
	imen M		.,	~ \				2		3	4	5		6
	ral P			07% )			.1.7	-						
				nches			4.7				····	-		
Area $D_{1AM} = 1.9^{\circ}$ sq. cms. Volume c. c. s.							18.4							
							219.7				· · · · · · · · · · · · · · · · · · ·			
Dry Unit Weight   Ibs/cu-ft- Gs= · Volume Soil Solids							80.0	•						
				r at		1	636 35							
	Tare +		V1 U 1 C	1 41	GIIU	-	635.31 531.7				i			
	ber an		aht of	Tare			249.9							
	Soil	u wer	giri oi	Ture			281.8							
		Weigh	t of	water			201.0	-			-			
Befo	re			ontent								+		
Tes	it			saturo								-		
		Weigh			111011		103.65	-						
Aft				ontent			36.8							
Tes	st			satura			36.6	/3						
		T T												
Load	Dial	Strain	Area	01	Load	Dial		Area	5.	Load	Dial	Strain	Area	51
Pan	Rdg	%	em2	Kgm/cm	Pan	Rdg				Pan	Rdg			
0	0530		10.7	,										
0gm 50	.0520		18.4											
100	.0126			1										
150	.9840	1.447	18.68	0.402										
200				0.532										
250	.9162									-				
300	.8750 .8305													
400	.7685													
450			1	1										
500	FAIL	D												
	1	1												
	1		}				-							
		1	1											
		1			Constant									
		- 1												
				-						-				

## UNIVERSITY OF PLBERTA 17.00 17.00 17.00 17.00 17.00 17.00 DEP T. OF CIVIL ENGINEERING TOTION OF SUL MECHANICS LABORATORY HONE BONDIAN STATE TO TRIBUIAL COMPRESSION Discription of something -oto() -ciliunM ol snidabM Wultiplication Factor\_\_\_\_\_ W. sading Block + Piston (gms)\_ ATAG SPECIMEN 6 d ES 1 Speciate Namber Lateral Pressure ( d. Length Ared Ties Volume c.c.c. Dry Unit Waight the/out! Gs= . Voirme Soil Sollies W- mre + fri - Well of tort 606 10 Tales + 1102 - 6107 117 Wt. Torn - Soll 0.0564 though a second of the Wh Swit Veight of mole Beior Maines anutaint PaT makerurue to estant Afrer Weight of well ... Test Moisture gentent Dedies of school 000 becl 600 1610 THIC Strain Argo en /Big. Senin Area BIRD MATIE nag nag 10 DAR 109 pos DOR 45 The 1250 SEEP THE REPORT OF LICENSE 26.0 7.0 103 213 213 study called distributed THE PARTY OF THE P The Bridge of the Paris TELLIS PINE

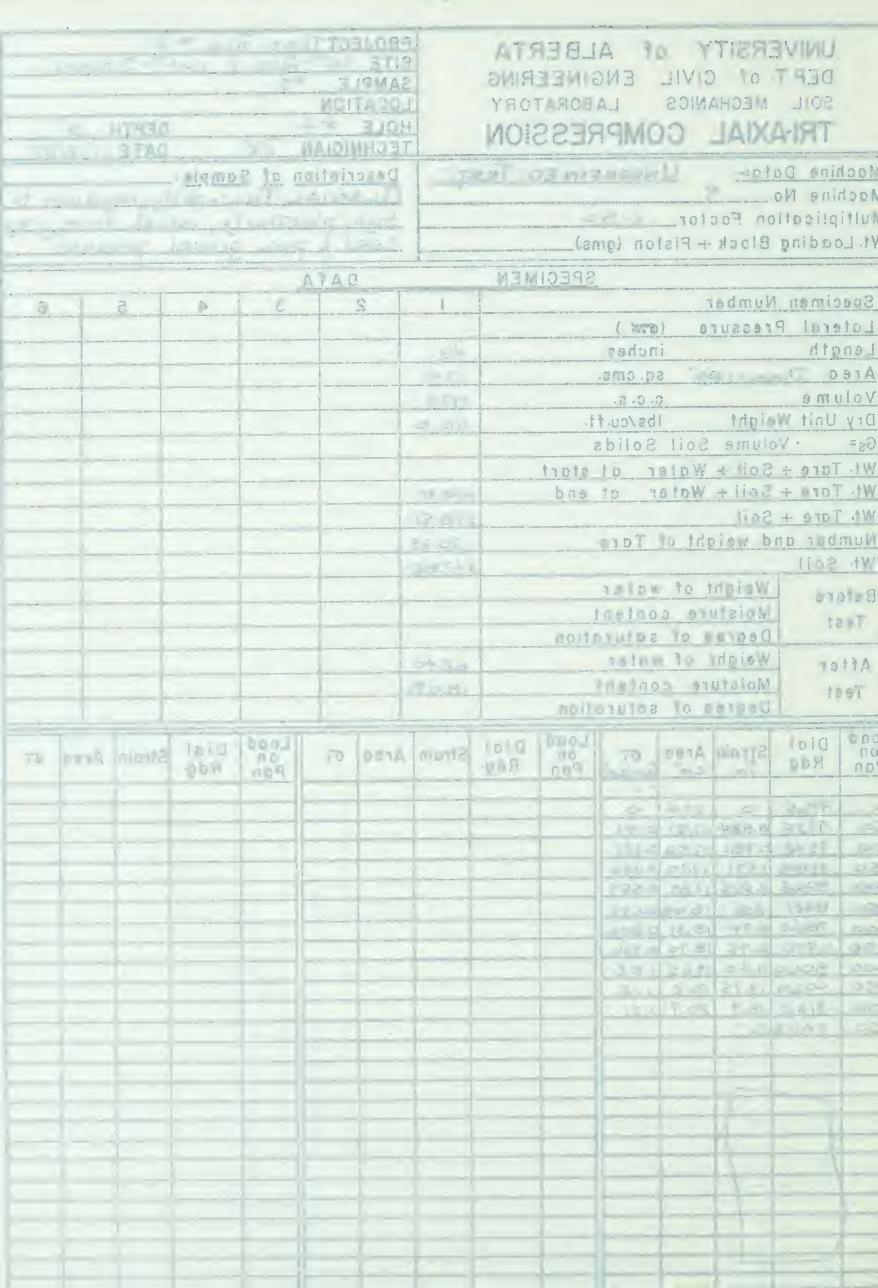
UNIVERSITY of ALBERTA						PF	ROJE	CTTE	STT	TE #	4				
DEPT of CIVIL ENGINEERING						SITE 114th AVE & 144th STREET.									
				LA					CAT		٠				
				OMPR					OLE	#	4		DEPTH	/31	
	IIIIA	NAL	U	PINIEL	<b>C</b> 33	IUI		TE	CHN	ICIAN	P.K.		DATE		
Mach	ine Da	ta:-	L	Heor	FINE	OTE	ST.					ample:			
	ne No		9									plast		oist.	
				X 5							col		E iii		,
Wt. Lo	ading	Block .	+ Pist	on (gm	s.)			-	-						
					SPECII	MEN			[	ATA					
Spec	imen 1	lumbei					1	T	2		3	4	5		6
Late	ral P	ressur	e (	67H )											
	1 th			nches			4.8								
		1m. = 1.	9" s	q. cms.			18.4								
	me			C. C. S.			224.	0							
	Dry Unit Weight   Ibs/cu-ft-						17.5						-		
	Gs= · Volume Soil Solids														
	Wt. Tare + Soil + Water at start Wt. Tare + Soil + Water at end						413.8								
	Tare +		11016	, 41	GIIU		308.8					<u> </u>			
	ber an		aht of	Tore			300.8								
	Soil	J HOI	4111 01	1016			218.0								
		Weigh	t of	water			7 7 6.0								
Befo			1	ontent											
Tes	1			saturo											
Aft	0.5	Weight					105.0	6							
				ontent			37.8°								
Tes	)			satura											
Load	Dial				Load	Dial		T			Load	Dial			
on Pan	Rdg	Strain	Area	Vomlen	Load on Pan	Rdg		n /	Area	0.	on Pan	Rdg.	Strain	Area	51
1 411	3	10	CM	Kgm/cm	1 411	- 3					, 411	4			
Ogm	.0259		18.4												
0gm 50	.0158							-							
100	.0000	0.539			-										
200	.9858	0.834	18.58	0.538											
250	9430	1668	D(NE	KINCY	-			18	3.74	0.667		<u> </u>			
300	1000.	1.875	18.77	0.800											
350	.6418	2.27	18.82	0.930				-							
400	.5840	2.15	19.1	1.18											
500	FAILE	1	1,.,	1.7.5											
	1	1													
	1	1							-						
	1	A			,										A S
		1						+							
	E .		1		1						1				



UNIVERSITY of ALBERTA								PROJE	CT	EST	PILE	#4		
DEPT of CIVIL ENGINEERING							SITE 114th AVE & 144th STREET. SAMPLE #4							
				LA				LOCAT		44				
								HOLE	#	4		DEDTU	10'	
	11717	AXIAL	. 00	OMPF	KESS	ION		TECHN			<del>/</del>	DEPTH	18	
Mach	ine Do	ita:-		UNCO	SNEIN	1507	EST						117/	27
				9.40.		IED I	<u>F91.</u>	Desc	ripilo	n of S	ample:	- 1	c+ 0	P
				x 50	`			100	-	night	y pla	stic,	2114-	1,
				on (gm				MOI	51,	grey	in co	dor.		-
17 1. L.C	duning	DIOCK	T FIS	ion (gm	18./									
					SPECI	MEN			DATA					
Spec	cimen	Numbe	r				1	2		3	4	5		6
Late	eral P	ressui	re (	67% )										
Len			-	nches			4.8							
		AM. = 1.		sq. cms.			16.4							
	ı m e	.,					200.							
				bs/cu-f	t.		82.2							
	Gs= · Volume Soil Solids						02.2							
				r át										
				r at			1153	7					-	
	Tare +		77 (31 (		UTIU		615.3 513.4							
			abt of	Tara										
		ia_wei	gni oi	Tare			249.9							
AA 1 ·	Soil	Watab	1 -6				263.4	74						
Befo	re			water										
Tes	st			ontent										
				saturo	TION			,						
Aft	er			water			101.9							
Tes	st			ontent			38.7	70						
		Degre	e of	satura	tion									
Load	Dial				Load	Dial	1			Load	Dial			
Pan	Rdg	Strain	Area	Kam/cm	on Pan	Rdg		Area	5.	Pan	Rdg	Strain	Area	51
		1-/3		3//5/										
Dams	.7262	0	16.4	0										
50	.7262	0.943	16.55	0.151										
100		5 1.930												
150		3.09												
200		5.63									· · · · · ·			
250 300		B.04												
350	FAIL													
				1										
				-										
_				1										

## UNIVERSITY OF ALBERTA SAMPLE DEPT. OF CIVIL ENGINEERING MOLTANO SOIL MECHANIS LABORATORY 3 (CH DEPTH TRHAXIAL COMPRESSION TE OHUI CIAN L. LAN E THE THE HEAVY Description of Samples - ptpl didooi and the same of th achine No. And the second s luttialization Factor\_\_\_\_ It Lording Block + Pi ton (ama) ATAC SPECIF EN 0 E S Succimen Number Lateral Pressure (umi) nohes Length sa.cms. E. O SA y olume 0.0.0 full Wight lbs/cuff 1 5 1 Go · Volume Soil Solids that to retail - line sent tw 10 20 4 Wit Tora - Soil + Waler al und 54.818 Wh Tare - Sall number and wolcht or Taxe HITE IW Watght of Worldw Bafare Instruct assisted 1281 Degree of relovation -# los asiow to indian 79. A Meisters redsigM VILOR. TEST Define of cottenion 0.00 0.00 0.00 0.000 bood 1010 Dig Diol TO COUTA PINTS Stroin NEED OF Strain Arno in digit ObA CERT nbH Pan 0.14 The state of the s The state was seen and arolly and color 12 0 12 13 1 He 10 11 14 E 1 2 E 1 SOUNDER.

UNIVERSITY of ALBERTA							SITE 14th ANE & 144th STREET.								
	DEP T. of CIVIL ENGINEERING							SAMPLE #5							
	SOIL							LOCAT		~					
	TRI-A							HOLE				DEPTH	23	/	
	IIIIA	MAL	- 00	א אועול	E33	IOIA						DATE			
Mach	ine Dat	a:-	U	NCONF	INED	TES					ample:		/		
Machi	ne No.							GLA	CIA	TIL	:- SI	ty me	910	m to	
	olicatio			x 50	>						y, mo				
Wt. Lo	ading	Block -	+ Pist	on (gm	s.)						ravel				
					SPECII	MEN			ATA		3				
Spec	imen N	lumbei					1	2		3	4	5		6	
Late	ral Pi	essur	e (	57H )											
Leng	th		i	nches			4-3								
Are	DIA	m. = 1.2	5" S	q. cms.			17.4								
Volu	m e		С	. C. S.			197.5								
	Unit We						110.0								
	· Vo														
Wt. 7	Wt. Tare + Soil + Water at start														
	Wt. Tare + Soil + Water at end						440.91								
_	Wt. Tare + Soil						378.51								
	ber an		tht of	Tare			30.6								
	Soil						347.80				-				
		Weigh	t of v	water											
Befo	16			ontent											
Tes		_	7	satura											
461		Weight					62.40								
Aft				ontent			18.09								
Tes				satura		<del></del>	16.07	3							
							1								
Load	Dial	Strain	Area	07	Load	Dial		Area	5.	Load	Dial	Strain	Are	0 51	
Pan	Rdg	%	cm2	(gm/em	Pan	Rdg				Pan	Rdg				
	GOL C		1	//											
50	.9965	0 5314	17.4												
100	9542														
150	.9288														
200	.8865														
250	8431														
300	.7840														
350	.6970														
450	.5060														
500	·3/3Z														
550	FAILE														
		-													
	-														
	/														
	<b>\</b>			-				-							
							-	-				1			



	UNIVERSITY OF ALBERT DEPT OF CIVIL ENGINEERS SOIL MECHANICS LABORATO TRI-AXIAL COMPRESSION							SITE SAMPL LOCAT HOLE	E #	+		DEPTH		
<u>Mach</u> Mach Multi;	ine Da ine No plicatio	ta:- - 9 on Fac	<u>\</u>	X50	YFINE		- <u>\$</u> 7.	Desci	ription CIA	n pla	ample: L:-d	ense san	, gr	ey,
					SPECII	MEN			ATA		<u> </u>			
Spec	imen 1	Numbe	r					1 2	-	3	4	5		6
	ral P			57K)										
Leng	gt h		i	nches			3.8							
Are	a Die	1m. = 1	.8"	sq. cms.			16.4							
	me			C. C. S.			158.2							
				lbs/cu-f			110.0		-					
·	Gs= ·Volume Soil Solids Wt· Tare + Soil + Water at start													
	Wt. Tare + Soil + Water at start Wt. Tare + Soil + Water at end							1						
	Wt. Tare + Soil + Water at end Wt. Tare + Soil						380.1				······································			
			ght of	fTare			48.1							
	Soil						279.24							
Befo	re	Weigh	t of	water										
Tes		Moist	ure c	ontent										
				saturo	ition									
Aft	er			water			52.84							
Tes	st			ontent			18.9 9	3						
	1	Degre	e of	satura	tion									
Load on Pan	Dial Rdg	Strain	Area	Kgmkm	Load on Pan	Dial Rdg.	Strain	Area	5.	Load on Pan	Dial Rdg.	Strain	Area	51
150	.875Z .875Z .8095	Br												
250	.6329		1				1	7 9						
300	FAILE													
	<u></u>													
							-							
				-										
									***************************************					
														1
7														
														(c. 50
		<del>                                     </del>												

PROJECT UNIVERSITY OF ALIBERTA DEPT. OF CIVIL ENGINEERING - 1914A. SO MICHANICS LA BORATORY MOTTABLE 41330 TRIAXIAL COMPRESSION TEC INCHAN \_\_L\_ATA/ DESI OIMENDONI Description of the lachine Dota: lachine No\_\_\_\_ A RELIGIOUS MELOSON ul iplication Factor The state of the s 't Loading Block + Piston (gms). DATA SPECIMEN 4\_ -Specimon Number Lateral Pressure (cm ) inches Lingth 3 \* ers J. . . . . sq. cms. ariatuavas are a-a-a .8.50 . muinl Dry Ull 'eight Instant Gas - Velume Boil Selids Wt. Tore - Soil + Wallet C stort M. Tore + Soil + Walor II ad CH TOK Number and selight of Taxe N. 0 1102 191 Weight of maler naotaB Moisture contain Vall T Dear of tolurchon Weight of weller - 3 1/2 TOITA Moleture souteful 29] Degras of saturation 500 Jial Shain Ared 0000 0007 1010 DIG Strain erea Stroin Arong 65 7% 312 100 10.50 Rdg Rulg gan 009 JTQ うしかぜい 2500 بند Ph 12 23 06 E 103 0120/12 L00

UNIVERSITY DEP'T of CIVI	L ENGIN	ERTA EERING RATORY	PROJECT TEST PILE #4.  SITE 114th AVE & 144th STREE  SAMPLE LOCATION						
MOISTURE	CON	TENT	TECHNI	#4 CIAN P.K	DEPT DATE	H 1/2/59			
Hole No.	4	4	4	4	4	4			
Depth	3	5'	8'	10'	13'	15'			
Sample No.	1		2		3				
Container No-	1A27	1423	1A27	1A35	1A86	1A53			
Wt. Sample Wet + Tare	54.48	81.61	63.27	89.44	56.91	82.27			
Wt Sample Dry + Tare	44.79	65.58	50.81	69.54	46.01	64.88			
Wt· Water	9.69	16.03	12.46	19.90	10.90	17.39			
Tare Container	17.67	17.27	17.67	17.71	17.38	17.55			
Wt. of Dry Soil	27.12	48.31	33.14	51.83	28.63	47.33			
Moisture Content w%	35.7	33.2	37.6	38.5	38.1	36.8			
Hole No	4	4	4	4					
Depth	18'	20	23'	25'					
Sample No	4		5	6					
Container No-	1A27	1A17	1A67	1A86					
Wt·Sample Wet + Tare	56.28	96.35	103.80	55.35					
Wt Sample Dry + Tare	45.91	75.20	90.07	49.66					
Wt· Water	10.37	21.15	13.13	5-69					
Tare Container	17.67	17.43	40.61	17.38					
Wt. of Dry Soil	28.24	57.77	49.46	32.28					
Moisture Content w%	36.4	36.6	27.8	17.6					
Hole No									
Depth									
Sample No									
Container No									
Wt Sample Wet + Tare									
Wt-Sample Dry + Tare									
Wt· Water									
Tare Container									
Wt. of Dry Soil									
Moisture Content w%									

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SOIL MECHANIC	Rosna E	VEDEN			P	reserves.					
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## LABORATORY TEST RESULTS

TEST PILE NO. 5



UNIVERSITY o	of ALB	ERTA	PROJECT TEST PILE # 5						
DEP'T of CIVIL	ENGIN	FERING	SITE 114th AUE & 144th STREET.						
SOIL MECHANICS			LOCATION						
			HOLE	# 5		DEPTH	4'		
ATTERBERG	LI	MITS	TECHNI	CIAN P.	۷.	DATE			
	Liq	uid Limit							
Trial No.		2	3	1	2		3		
No. of Blows	40	41	40	15	14		14		
Container No.	V36	441	V46	V19	V2<		V71		
Wt Sample Wet + Tare	2.0120	84.3073	99.4312	93.5888	79.47	90 8	36.2867		
Wt Sample Dry + Tare 7	15.5530		92.6951	86.9911	72.95	27	79.6222		
Wt· Water	6.4590	6.3368	6.7361	6.5911	6.5	263	6.6645		
	57.0527	69.6448	83.7237	79.5047	65.5	706	72.0433		
	8.5003	8.3257	8.9714	7.4869			7.5789		
Moisture Content w%	76.0	76.2	75.3	88.2	88	4	88.0		
	Aver	age Values		Plastic	Limit				
			Trial No.		l	Z	3		
	W	= 81.5 %	Container	No.	4	7	11		
	w	= 27.3 %	Wt. Sample	Wet+Tare	32.7891	33.27	06 42.3390		
	w <sub>s</sub>		Wt. Sample	Dry + Tare	32.1599	32.56	0441.6505		
	THE S	511-7 7	Wt. Water				2 0.6885		
	Ip.	= 54.2 %	Tare Cont				1439.1062		
	If		Wt of Dry				10 2.5443		
	I+	= 2.34	Moisture C			27.0	27./		
			H	Shrinkage	Limit				
			Trial No.						
			Container						
90			Wt. Sample						
			Wt Sample	Dry + lare					
8			Wt-Water	_•_					
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UNIVERSITY OF ALBERTA
DEP'T OF CIVIL ENGINEERING
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UNIVERSITY	UNIVERSITY of ALBERTA PROJECT TEST PILE #5									
DEP'T of CIVI				SITE 114th AVE & 144th STREET.						
SOIL MECHANIC			SAMPLE							
			HOLE	#5	D	EPTH	9'			
ATTERBER	G LI	IVII I 5	TECHNI	CIAN P. K		ATE I				
	Liq	uid Limit								
Trial No.	1	2	3	1	2		3			
No. of Blows	31	30	30	/1	11		12			
Container No.	1	V50	A4	Z	¥70	А	12			
Wt·Sample Wet + Tare	88.2527	94.5766	91.05/8	87.2670	98.74	96 90.	9610			
Wt·Sample Dry + Tare	82.5995	89.4086	85.3197	82.0701	91.756	64 87	.6886			
Wt. Water	5.6532	5.1680	5.7321	5.1969	6.993	32 8	2724			
Tare Container	74:0352	81.5525	76.5604	74.9626	82.148	B3 71.	3514			
Wt of Dry Soil	8.5643	7.8561	8.7593	7.1075	9.608	8/ 1/.	3372			
Moisture Content w%	66.0	65.7	65.2	73.2	72.9		73.0			
	Aver	age Values		Plastic L	imit					
		= 6b.9 %	Trial No.		1	Z	3			
	W W	= 6p. 1/0	Container	No.		Z	3			
	w	28.3 %	Wt. Sample	Wet+Tare 4	9.9075	50.59/3	46.8976			
	$w_{\rm s}$	=	Wt. Sample		1					
			Wi Water		0.6/30					
	<sup>1</sup> p	= 38.6%	Tare Cont	ainer 5			43.8264			
74	4	<b>17.9</b>	Wt of Dry	Soil			2.3850			
	I,	= 2.16	Moisture C		28.1	28.2	28.7			
				Shrinkage	Limit		1			
73			Trial No.							
			Container							
12			Wt Sample							
			Wt Sample	Dry 1 Tare						
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TOBLODE UNIVERSITY OF ALBERTA 61 / F / I DEP'T OF CIVIL ENGINEERING SMPLE LOSSATION MROTAROSAL COMMINGE THE CEPTH : LOUIS | LOUIS LIMITS ATTERBERG Limid blunia OM 10 AF BIDWE old seeints Sample Well+ Tore Sminple Dry + Tore LOCAL THE Wild 28 1 1 and was the HITOW 3 LAS. W. 42-1-Y WOULD BE 23.00-13 536045 Assistant sy of Dry Soil 15 P. L. S 50 m 3 -3- 191 iffur inethed stured Plastia Limit Average Veluer Jaiot No. Transfer spir Gentelmar War T 1 85 100 We Sample Wath Tare In the Sample of the WI Sample Dry 1 Tare well to 181 0 Toninthoo sant 50 -1 We of Dry Soil Molafure Contact W | C | C | 210 : 1 Shrinkage Limit -016 1611T own nenthings WY-Sample Wet + Ture W Sample Dry + Tare TAXBW-LW TOUR CONTRIBUTE W Hos gro to IW Worlder Content w"5/ V tenintend lay Vely Dry Soil Pal Vs Strillinkogo Vol. V-V-Pur tords appaintme (00) + N-V) -w = 10 Description of Sumple Charle Land The Land Ramoras 05 35 615 Number of Blanc

UNIVERSITY of AL	BERTA	PROJECT TEST PILE #5					
DEP'T of CIVIL ENG		SITE 114th AVE & 144th STREET.					
		SAMPLE					
SOIL MECHANICS LAND		LOCATIO			POTH		
ATTERBERG	IMITS	TECHNI	CIAN P. K		DEPTH DATE /	12/59	
	iquid Limit	11201111	7.2			7937	
Trial No.	2	3	1	2		3	
No. of Blows 29	27	28	/3	/3		/3	
Container No. Yza	V36	¥71	¥79	741	\	146	
Wt Sample Wet + Tare 88.1863	87.3462	90.2118	102.9365	94.305	58 100	0.7643	
Wt. Sample Dry + Tare 79.206	7 79.3354	83.0734	93.2599			7.2781	
Wt Water 8.979	8.0108	7.1384	9.6766	10.10	3/ 9	7.5362	
Tare Container 65.570	6 67.0527	72.0433	19.5047	69.64	48 8	3.7237	
Wt. of Dry Soil 13.636	1 12.2827	11.0301	13.7552	14.55	79 1	3.5044	
Moisture Content w% 65.8	65.4	64.9	70.4	69.6	0	70.6	
	ange Values		Plastic L	imit			
	erage Values	Trial No.		1	2	3	
	Wz= 66.0 %	Container	No.	4	7	11	
	Wp= 29.6 %	Wt. Sample	Wet+Tare 3	2.9343	3z.873°	4.8710	
	P	Wt. Sample	Dry+Tare 3	2.2399	32.197	41.2407	
	Vs =	1000		. 1		0.6303	
	1p = 36.4 %	Tare Cont	ainer Z	9.8903	29.9014	139.1062	
	4 = 14.7	Wt of Dry	Soil .	2.3496	2.2954	2.1345	
	1 = 2.48	Moisture C	ontent %	29.6	29.5	29.6	
	11		Shrinkage	Limit			
11		Trial No.					
		Container	No.				
		Wt. Sample	Wet + Tare				
10		Wt Sample					
		Wt Water					
869		Tare Cont	ainer				
		Wt. of Dry	Soil Wo				
÷69		Moisture Co	ontent w%				
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UNIVERSITY of ALBERTA PROJECT TEST PILE #5											
	DEP'T OF CIVIL ENCINECONO SILE 14TH AVE & 144TH STREET.										
SOIL MECHANIC	S LARO	RATORY	LOCATION								
ATTERBER		MITS	HOLE	#5		DEPTH	19'				
ATTENDEN	O LI	IVII I S	TECHNI	CIAN P.			1/20/59				
	Liq	uid Limit					1 1				
Trial No.	1	2	3	/	2		3				
No. of Blows	30	30	28	14	/3		14				
Container No.	Y20	V36	141	V46	V19		V 7/				
Wt. Sample Wet + Tare	87.7516	91.9382	93.5175	109.3431			18.6826				
Wt. Sample Dry + Tare	78.4710	81.5530	83.5179	98.074			87.0067				
Wt. Water	9.2806		9.9996	11.2686			11.6759				
Tare Container	65.5706		69.6448	83.7237			72.0433				
Wt. of Dry Soil	12.9004	14.5003	13.8731	14.3508			14.9634				
Moisture Content w%	72.0	71.6	12. Z	77.9							
				Plastic	Limit						
		age Values	Trial No.		/	Z	3				
	w	= 13.2 %	Container	No.	4	7	1/				
	w.	= 25.6 %	Wt. Sample	Wet+Tore		32.424					
			Wt. Sample								
	ws		Wt. Water				9 0.5209				
	Ip	= 47.6 %	Tare Cont				439.1062				
79	I <sub>4</sub>	= 19.2	Wt. of Dry				7 2.0138				
			Moisture Co	ontent %	25.6						
	-1			Shrinkage							
78			Trial No.								
			Container	No.							
77			Wt.Sample \	Wet + Tare							
			Wt Sample [	ry + Tare							
%			Wt Water								
76			Tare Cont	giner							
-			Wt. of Dry S	oil Wo							
t e u			Moisture Co	ntent w%							
v 75 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0			Vol. Contain								
ပိ			Vol. Dry Soi		3						
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71				<del></del>							
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TOFFICE UNVERSITY OF AUBERTA DEP'T OF GIVIL SHIBBERNE BY 3181475 MOITSOO LABORATORY COLL MECHANICS 31 AC LIMITS ATTERBERG Liquid Limit .611 10 e=018 to . plainer Nu , - Edmple Wet + Tore SHOP IP 10230 2-10-87 OX/2 IB Sample Dry 4 Tore W2.02.5 The same by the SEREN! Notok : 533000 1200 10 10018 10 te Contouner HOZ VYG TO A A 5.5 Star Institut Cantoni was timel Dironia Anyorage Values -0.00\_0.0(xT S' Tell open Contoiner No-7 432 300 The La Bollon, and Strott Wolf of Comp. 17W. wy Sample Dry Ploto Errage Survey 121 Const - 1 to 5 to 5 ARREW JW JAK . O 11.8101 to 101 to 101 to Tore Lealmant - - -ALIGN SMI LINE AND EAST AND THE AN Dolphya Content "by 25 b 25 b 25 b 6.501 Shrinange Limin -- OM PRINT OVI TENIUMET NO BYOT FIRM ALTIMOSTE AYOTH VALUE OF COMES The same of the sa rentermant anny W. of Ory Sail 14. Moleture Content Airs. Val- Carronary A TOP NOT END My Strinkung Vol. V-V. 2- iimid spakimi (aor + -11-1) -w = 31 Description of somplet RESTROKES: 04 DE 25 09

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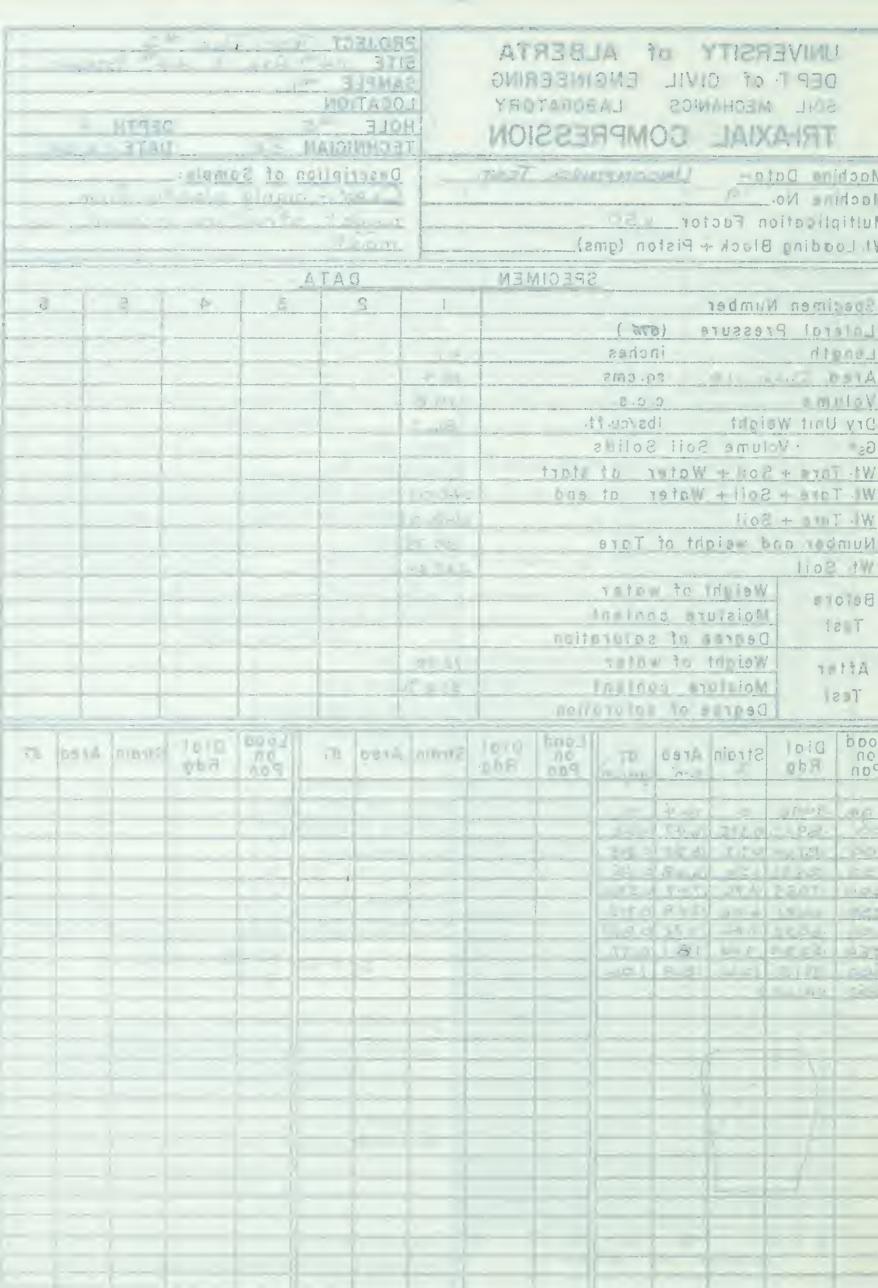
UNIVERSITY of AL	BERTA	PROJEC	T TEST 7	TLE #.	5		
	NEERING	SITE	- +-				
		SAMPLE					
SOIL MECHANICS LAB		LOCATION			DEPT	<b>L</b> 2	11!
ATTERBERG L	.IMITS	TECHNI					30/59
L	quid Limit	11201111	OTAIL TIX			-//-	30/3/
Trial No.	Z	3	/	Z			3
No. of Blows 43	4-2	44	20	20		Z	:/
Container No. V79	V36	Y71	V41	V41	0		20
Wt. Sample Wet + Tare 106.794:			96.2995				1273
Wt Sample Dry + Tare 97.998		92.438Z					.8403
Wt. Water 8.796		9.5078		9.19			.2870
Tare Container 79.504		72.0433					.5706
Wt. of Dry Soil 18.4930			17.6536				.2697
Moisture Content w% 41.6	47.4	46.8	51.1	51.1			0.7
			Plastic L	imit			
	erage Values	Trial No.	1 1 1 1 1 1	/	2		3
1	= 49.9 %	Container	No.	4	7		11
	p= 26.5 %				336	356	
		Wt. Sample	Dry+Tare				
	/s =	WA Mator					0.5984
	p = 23.4 %	Tare Conf					39.1062
	* 11.6	Wt. of Dry					2.2825
	•	Moisture C		26.6	26.		26.3
	= 2.02	4	Shrinkage	Limit			
52		Trial No.					
		Container	No-				
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8 50		Tare Con	tainer				
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Moisture 141			, - 60 (	Wo			
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UNIVERSITY OF ALBERTA DEP'T OF CIVIL ENGINEERING SOIL MECHANICS LABORATORY

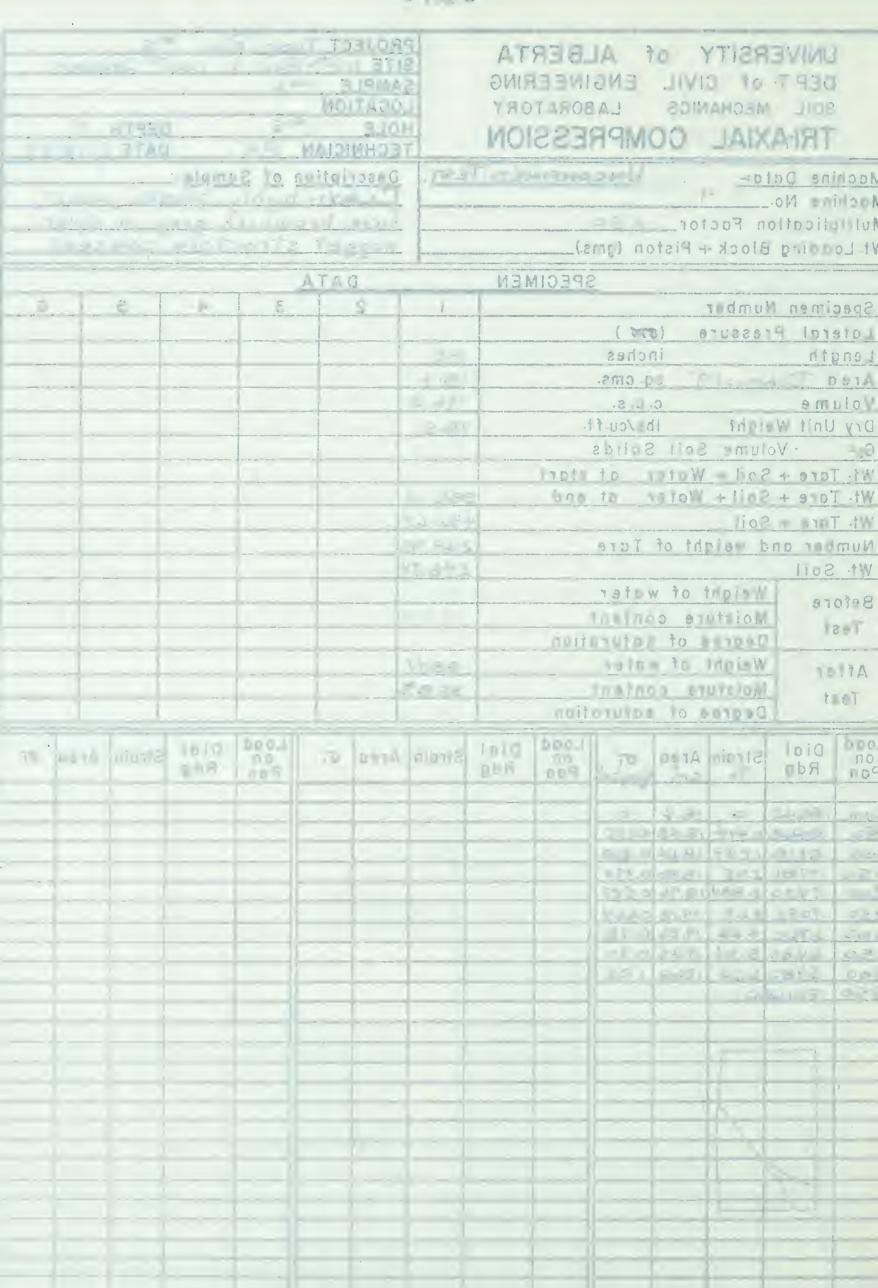
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	UNIVE DEP TO SOIL TRI-A	of (	CIVIL	ENG LA	INEE BORAT	RING		SAMPL LOCAT HOLE	E # ION #5		PILE =	DEPTH	<i>#</i> ′		
	ine Da		UN	CONFI	NED	TEST		Desc	ription	n of S	ample:		,		
	ine No							CLA	x:- 1	nghly	plas	tic fi	rm.		
	plication							nogo	et	struct	ture :	rese	nt qu	rev	
Wt. Lo	pading	Block	+ Pist	on (gm	s.)			mugget structure present, grey							
					SPECI	MEN		[	DATA						
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Load on Pan	Dial Rdg	Degre Strain	Area	satura or kgm/cm	Load on		Strain		67	on		Strain	Area	<i>σ</i> τ	
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Load on Pan	Dial Rdg .9092	Strain O 0.395	Area cm	satura or kgm/cm o o.152	Load on		Strain		67	on		Strain	Area	στ	
Load on Pan	Dial Rdg .9092 .8930 .8700	Strain  O  0.395  0.713  1.56	Area Cm 16.4 16.47 16.52 16.68	67 Kgm/cm <sup>2</sup> 0.152 0.303 0.45	Load on		Strain		671	on		Strain	Area	στ	
Load on Pan Ogm 50 100 150 200	Dial Rdg .9092 .8930 .8700 .8451	Strain % 0.395 0.713 1.56 4.96	Area Cm2 16.4 16.47 16.52 16.68	50 tura 60 to 152 0.303 0.45 0.586	Load on		Strain		67	on		Strain	Area	στ	
Load on Pan Ogm 50 100 150 200 250	Dial Rdg .9092 .8930 .8700 .8451	Strain % 0.395 0.713 1.56 4.96 6.02	Area Cm 16.4 16.47 16.52 16.68 17.09	67 Kgm/cm 0 0.152 0.303 0.45 0.586 0.715	Load on		Strain		Gi	on		Strain	Area	67	
Load on Pan Ogm 50 100 150 200 250 300	Dial Rdg .9092 .8930 .8700 .8451 .7059 .6621	Degre Strain % 0.395 0.713 1.56 4.96 6.02 7.46	Area Cm 16.4 16.47 16.52 16.68 17.09 17.48 17.72	50 tura 6 m/cm 0 0.15 z 0.30 3 0.45 0.58 b 0.715 0.847	Load on		Strain		671	on		Strain	Area	61	
Load on Pan Ogm 50 100 150 200 250 350 400	Dial Rdg .9092 .8930 .8700 .8451 .7059 .6621 .6032 .5230	Degre Strain % 0 0.395 0.713 1.56 4.96 6.02 7.46 9.43 12.61	Area Cm 16.4 16.47 16.52 16.68 17.09 17.48 17.72	07 Kgm/cm 0.152 0.303 0.45 0.586 0.715 0.847 0.97	Load on		Strain		61	on		Strain	Area	σι	
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Load on Pan Ogm 50 100 150 200 250 350 400	Dial Rdg .9092 .8930 .8700 .8451 .7059 .6621 .6032 .5230	Degre Strain % 0 0.395 0.713 1.56 4.96 6.02 7.46 9.43 12.61	16.4 16.47 16.52 16.68 17.09 17.48 17.72	07 Kgm/cm 0.152 0.303 0.45 0.586 0.715 0.847 0.97	Load on		Strain		671	on		Strain	Area	στ	
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Load on Pan Ogm 50 100 150 200 250 350 400	Dial Rdg .9092 .8930 .8700 .8451 .7059 .6621 .6032 .5230	Degre Strain % 0 0.395 0.713 1.56 4.96 6.02 7.46 9.43 12.61	16.4 16.47 16.52 16.68 17.09 17.48 17.72	07 Kgm/cm 0.152 0.303 0.45 0.586 0.715 0.847 0.97	Load on		Strain		51	on		Strain	Area	σι	
Load on Pan Ogm 50 100 150 200 250 350 400	Dial Rdg .9092 .8930 .8700 .8451 .7059 .6621 .6032 .5230	Degre Strain % 0 0.395 0.713 1.56 4.96 6.02 7.46 9.43 12.61	16.4 16.47 16.52 16.68 17.09 17.48 17.72	07 Kgm/cm 0.152 0.303 0.45 0.586 0.715 0.847 0.97	Load on		Strain		61	on		Strain	Area	σι	
Load on Pan Ogm 50 100 150 200 250 350 400	Dial Rdg .9092 .8930 .8700 .8451 .7059 .6621 .6032 .5230	Degre Strain 0 0.395 0.713 1.56 4.96 6.02 7.46 9.43 12.61	16.4 16.47 16.52 16.68 17.09 17.48 17.72	07 Kgm/cm 0.152 0.303 0.45 0.586 0.715 0.847 0.97	Load on		Strain		61	on		Strain	Area	σι	
Load on Pan Ogm 50 100 150 200 250 350 400	Dial Rdg .9092 .8930 .8700 .8451 .7059 .6621 .6032 .5230	Degre Strain 0 0.395 0.713 1.56 4.96 6.02 7.46 9.43 12.61	16.4 16.47 16.52 16.68 17.09 17.48 17.72	07 Kgm/cm 0.152 0.303 0.45 0.586 0.715 0.847 0.97	Load on		Strain		61	on		Strain	Area	57	
Load on Pan Ogm 50 100 150 200 250 350 400	Dial Rdg .9092 .8930 .8700 .8451 .7059 .6621 .6032 .5230	Degre Strain 0 0.395 0.713 1.56 4.96 6.02 7.46 9.43 12.61	16.4 16.47 16.52 16.68 17.09 17.48 17.72	07 Kgm/cm 0.152 0.303 0.45 0.586 0.715 0.847 0.97	Load on		Strain		51	on		Strain	Area	51	
Load on Pan Ogm 50 100 150 200 250 350 400	Dial Rdg .9092 .8930 .8700 .8451 .7059 .6621 .6032 .5230	Degre Strain 0 0.395 0.713 1.56 4.96 6.02 7.46 9.43 12.61	16.4 16.47 16.52 16.68 17.09 17.48 17.72	07 Kgm/cm 0.152 0.303 0.45 0.586 0.715 0.847 0.97	Load on		Strain		51	on		Strain	Area	51	
Load on Pan Ogm 50 100 150 200 250 350 400	Dial Rdg .9092 .8930 .8700 .8451 .7059 .6621 .6032 .5230	Degre Strain 0 0.395 0.713 1.56 4.96 6.02 7.46 9.43 12.61	16.4 16.47 16.52 16.68 17.09 17.48 17.72	07 Kgm/cm 0.152 0.303 0.45 0.586 0.715 0.847 0.97	Load on		Strain		61	on		Strain	Area	σι	
Load on Pan Ogm 50 100 150 200 250 350 400	Dial Rdg .9092 .8930 .8700 .8451 .7059 .6621 .6032 .5230	Degre Strain 0 0.395 0.713 1.56 4.96 6.02 7.46 9.43 12.61	16.4 16.47 16.52 16.68 17.09 17.48 17.72	07 Kgm/cm 0.152 0.303 0.45 0.586 0.715 0.847 0.97	Load on		Strain		61	on		Strain	Area	51	
Load on Pan Ogm 50 100 150 200 250 350 400	Dial Rdg .9092 .8930 .8700 .8451 .7059 .6621 .6032 .5230	Degre Strain 0 0.395 0.713 1.56 4.96 6.02 7.46 9.43 12.61	16.4 16.47 16.52 16.68 17.09 17.48 17.72	07 Kgm/cm 0.152 0.303 0.45 0.586 0.715 0.847 0.97	Load on		Strain		51	on		Strain	Area	σι	



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Pan	Dial Rdg	Strain	Area	or Kgm/cm	Load			Area	o.	on		Strain	Area 67	
Pan	Dial Rdg	Strain	Area cm	or Kgm/cm	Load			Area	5.	on		Strain	Area or	
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Pan So 100	Dial Rdg .8642 .8468	Strain % 0 0,414 1.249	Area cm 18.4 18.48 18.64	0.135 0.268	Load			Area	5.	on		Strain	Area or	
0gm 50 100	Dial Rdg .8642 .8468 .8118	Strain % 0 0.414 1.249 2.05	Area cm 18.4 18.48 18.64	0.135 0.268	Load			Area	5.	on		Strain	Area or	
Pan 50 100 150 200	Dial Rdg .864z .9468 .8118 .7181	Strain % 0 0.414 1.249 2.05 2.884	Area cm 18.4 18.48 18.64 18.80	0.135 0.268 0.399 0.527	Load			Area	σί	on		Strain	Area or	
Pan 50 100 150 200	Dial Rdg .8642 .8468 .8118 .7181 .7181 .7430	Strain % 0,414 1.249 2.05 2.884 3.69	Area cm 18.4 18.48 18.64 18.80 18.96	0.135 0.268 0.399 0.527 0.654	Load			Area	σ.	on		Strain	Area or	
Pan 50 100 150 200 250 300	Dial Rdg .8642 .8468 .8118 .7181 .7181 .7430	Strain % 0,414 1.249 2.05 2.884 3.69 4.48	Area cm 18.4 18.48 18.64 18.96 19.72 19.72	0.135 0.268 0.399 0.527 0.654 0.78	Load			Area	σ.	on		Strain	Area or	
Pan 50 150 250 250 350 400	Dial Rdg .8642 .8468 .8118 .7181 .7430 .7092 .6760 .5980	Strain % 0.414 1.249 2.05 2.884 3.69 4.48 5.34 6.34	Area cm 18.4 18.48 18.64 18.80 18.96 19.72 19.72 19.75	0.135 0.135 0.268 0.399 0.527 0.654 0.78	Load			Area	σί	on		Strain	Area or	
Pan 50 150 250 250 350 400	Dial Rdg .8642 .8468 .8118 .7781 .7430 .7092 .6760	Strain % 0.414 1.249 2.05 2.884 3.69 4.48 5.34 6.34	Area cm 18.4 18.48 18.64 18.80 18.96 19.72 19.72 19.75	0.135 0.135 0.268 0.399 0.527 0.654 0.78	Load			Area	5.	on		Strain	Area or	
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## UNIVERSITY OF ALBERTA DEPT: of CIVIL ENGINEERING LABORATORY SOIL MECHENICS THAXIAL COMPRESSION

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